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ISSUED QUARTERLY

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THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 900 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

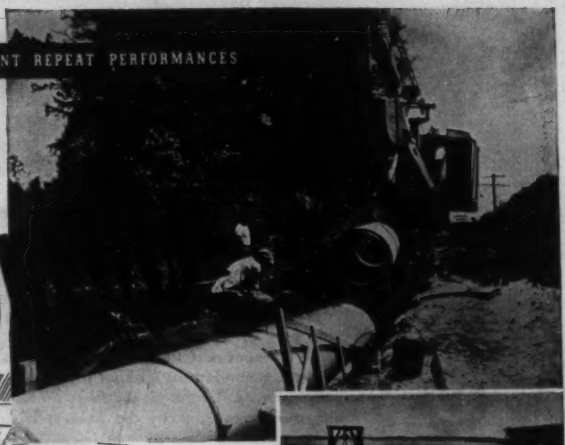
A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

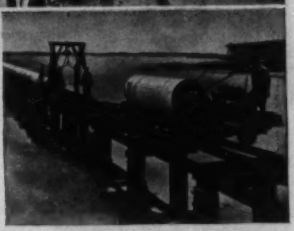
The initiation fees and annual dues are as follows:

	Initiation Fees	*Annual Dues*
Members	$ 3.00	$ 6.00
Juniors	1.00	3.00
Associates	10.00	10.00
Corporate Members	10.00	10.00

This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September on such date as the Executive Committee may designate.

LOCK JOINT REPEAT PERFORMANCES





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1943 – (Top photo) Installing 36" pipe alongside of old wood stave pipeline which it replaces.

1921 – (Lower photo) Installation of Lock Joint pressure pipeline on Trestle over Elizabeth River.

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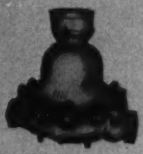

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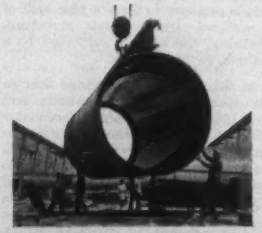

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Engineering Facts about
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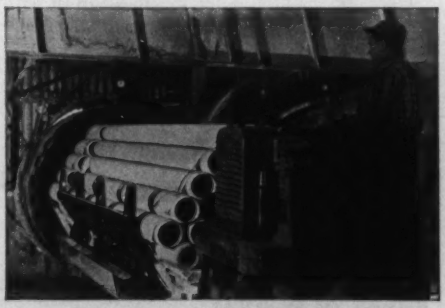
Resistance to Corrosion ... an index of long life

Ability to withstand corrosion is the most important single measure of the durability or life expectancy of an underground water pipe material. Two factors—both inherent in the pipe itself—contribute to Transite's exceptional ability to resist corrosion.

These are:

1. The inherently corrosion-resistant materials of which Transite is made.
2. The specially developed manufacturing process — employed exclusively by Johns-Manville—which imparts a high degree of chemical stability to the finished product.

In the manufacture of Transite Pipe, the three basic ingredients, asbestos fibres, cement, and silica—all basically corrosion-resistant by nature—are consolidated under tremendous pressure to form a pipe wall of dense, uniform, homogeneous structure. After forming, the pipe is subjected to a special steam curing process.



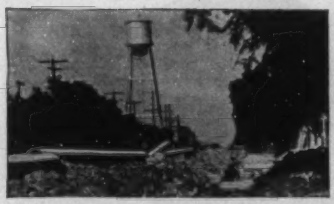
A load of Transite Pipe about to enter the steam curing tanks. This step in the Johns-Manville manufacturing process contributes substantially to the corrosion resistance of the finished pipe—and, therefore, to its long service life.

Transite Pipe was first used by this large west coast city in 1933. Its exceptional corrosion resistance—an index of long life—has already made it possible for Transite to outlive other pipe several times over.

*Transite is a registered Johns-Manville trade mark for its asbestos-cement pipe.

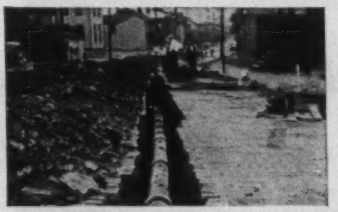
It is in this steam curing stage that so much is contributed to the stability and structural integrity of the pipe. Here under the action of pressure steam, Transite assumes a new chemical identity. The silica unites chemically with the free lime ordinarily associated with cement products and converts it into highly stable, insoluble calcium silicates. As a result, the cured pipe is unusually resistant to corrosive attack throughout its entire structure.

This intrinsic resistance to corrosion has been substantiated by numerous installa-



Transite Pipe was installed in this Texas city ten years ago to replace another pipe material that had been destroyed by soil corrosion in 7 years. The Transite mains are still on the job today with a long useful life ahead of them.

tions. Some of these have been exposed to highly aggressive soils, both alkaline and acid, for many years. Many are now serving as replacements for other types of pipe materials under conditions so destructive that their useful service life had been seriously curtailed.



Like thousands of other communities, this West Virginia city selected Transite Pipe because it promised assurance of maximum life. Today, after 14 years of service, the first installation of Transite has already fulfilled this promise by outlasting the pipe material previously used.

In one such installation, a Transite main installed during 1932 in an extremely corrosive soil was recently made the subject of careful study to determine its condition. Sections of the pipe, including couplings, were dug up and shipped to the factory for test. There was no evidence of deterioration. Pipe and couplings readily withstood the original factory test, equivalent to four times the normal working pressure of the line.

Certain types of industrial service provide an even more severe "proving ground" for the life expectancy of pipe materials, and here, too, Transite Pipe has demonstrated exceptional corrosion resistance. Coal mine service is a typical example. Here acid mine waters are frequently so corrosive that they have destroyed ordinary pipe materials in a matter of a few weeks or months. Yet Transite Pipe has handled these same waters under working pressure up to 150 lbs. for periods from 10 to 15 years with little, if any indication of deterioration.

To evaluate the ability of pipe materials to withstand soil corrosion, the National

Corrosive soil conditions were so severe at this location in a prominent New England city that the life of the pipe material formerly used was only 15 years. Transite Pipe, put in as a replacement in 1934, continues to give the same efficient, dependable service as the day it was installed.

Bureau of Standards has conducted an extensive series of field tests. These studies are based on examination of hundreds of pipe samples periodically removed from severely corrosive soils. In these and similar tests, Transite Pipe has consistently demonstrated its superior resistance to soil corrosion, confirming the long life expectancy which this asbestos-cement pipe has evidenced in thousands of water works installations.

Transite's ability to provide long-term, dependable service is well illustrated by its performance in coal mines, where it consistently outlasts other pipe materials in carrying corrosive mine drainage waters. The 36" Transite line shown above has been conveying acid mine waters for 15 years.

For further details about Transite Pressure Pipe, write Johns-Manville, Box 290, New York 16, N. Y.

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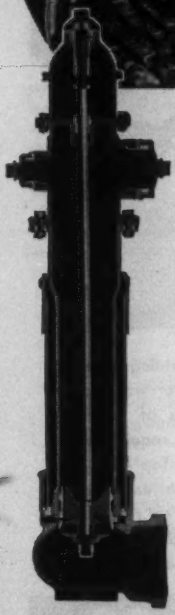

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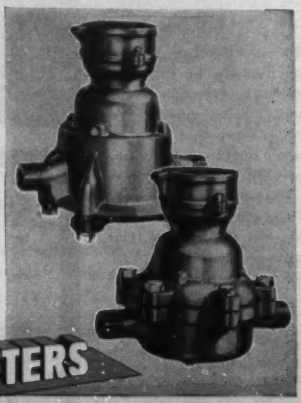

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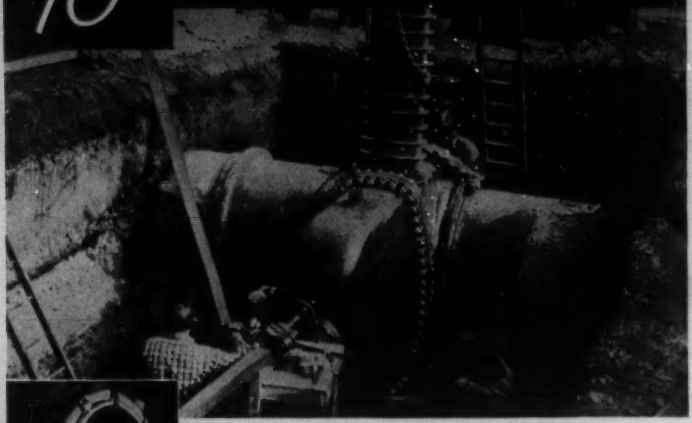

CUTTING THE MAIN. Illustration shows cutting dome, temporary shut-off valve and permanent valve body in place.

Section removed from main

Finished insertion

At a large Eastern steel mill uninterrupted production depended upon a thirty year old, 48" cast iron main, carrying sea water used for cooling the furnace jackets. The main was in poor condition — badly corroded and electrolytically pitted.

In order to provide an emergency by-pass in the event of line failure, a gate valve was required. Since the mill could not risk even a momentary shut down, Smith was asked to insert the valve under pressure. Although of unprecedented size, the insertion was completed in four days without interruption of service or reduction in pressure.

*First 48" insertion ever made under pressure. One month later a second 48" gate valve was inserted by Smith for the Water Bureau, Philadelphia, Pa.

THE A. P. SMITH MFG. CO.

ESTABLISHED 1896

EAST ORANGE, NEW JERSEY

Table of Contents

PAGE

Modern Diesel Practice for Water-Works Plants
By David L. Gallagher 105

Diesel Pumping Unit for Manchester Water Works
By Clarence L. Ahlgren 117

Introduction of Pre-chlorination of Slow Sand Filters
By Samuel Jacobson and Marshall S. Wellington 128

One Hundred Years of Public Water Supply for Boston
By James J. Matera 150

Algae Control at Danbury, Connecticut
By Ellis A. Tarlton 165

Report of Committee for Survey of Ground-Water Supplies in New England 175

Proceedings:

February 1949 Meeting 201

March 1949 Meeting 201

Roster of Sanitary and Public Health Engineers 206

New England Water Works Association
ORGANIZED 1882

VOL. LXIII JUNE, 1949 NO. 2

This Association, as a body, is not responsible for the statements or opinions of any of its members

MODERN DIESEL PRACTICE FOR WATER-WORKS PLANTS

BY DAVID L. GALLAGHER*

[*Read September 15, 1948.*]

TWENTY-FIVE years ago the use of Diesel engines was a new application, a new era in water-works practice. Diesel engines were used to drive reciprocating pumps as well as centrifugal pumps. It would probably be more correct to say centrifugal as well as reciprocating, for in 1923 most water-works pumping was done by reciprocating pumps.

Today, 25 years later, the Diesel engine is used to drive centrifugal pumps and generators. Since most water works in the New England area have low-price electricity available, Diesel-engine-driven pumps usually are installed as standby units, but this does not detract from the fact that the Diesel engine has proven to be a thoroughly dependable, continuous-duty driver, as well as a most economical one. Its outstanding additional advantage is its ability to be put into service in a matter of minutes, in the event of an emergency. We need only to mention in passing the large number of Diesel engines used in the marine and locomotive fields, as well as those generating power in rural electrification areas, as evidence of their dependability.

Stationary-type Diesel engines usually operate at considerably slower speed than the centrifugal pumps which they drive and, in these cases, a speed increaser is used, to obtain the pump speed desired. Gear sets are used not only for horizontal pumps but they are also available in the right-angle type, for use with vertical pumps.

A Diesel-engine-driven centrifugal pump has an inherent ad-

*Public Works Division, Worthington Pump & Machinery Corp., Harrison, N. J.

vantage in the ease with which variable-speed operation may be obtained. A simple adjustment, manual or automatic, controlling the amount of fuel oil injected, permits sufficient change in the pump speed to provide for varying system demands.

To illustrate most clearly the modern trend in water-works pumping, let us trace the history of one of your own plants—namely, that of the city of Fall River, Mass. Their source of water is the same now as when they started in 1874—74 years ago—the North Watuppa Pond, which provides a storage capacity of 7¼ billion gallons. The quality of the water is excellent, only a small amount of chlorine being required for treatment.

Fall River has no high-level storage reservoir and therefore has installed a number of large-capacity storage tanks in various parts of the city. Two tanks of 1 1/3 mil gal capacity each are located on a hill close by the pumping station; three other tanks, each of 1 2/3 mil gal capacity, are located around the outskirts of the city. This gives a storage capacity of more than 7½ mil gal today.

The first unit was installed in 1875. This was a Worthington, 5-mgd, steam-engine-driven reciprocating pump. From time to time other steam-engine-driven units were installed. In 1916, Fall River installed its first electric-motor-driven centrifugal pump—an 8-mgd, 350-hp, squirrel-cage motor-driven unit. Since 1916 steam has been only a secondary source of power.

In 1928 and 1945 additional motor-driven units were installed. In both cases they were Worthington, 18-LA-3, 12-mgd, 500-hp synchronous-motor-driven pumps. The original 8-mgd motor-driven pump, installed in 1916, has been removed and the two 12-mgd units carry the load, now averaging 9 mgd.

For standby service there were two steam-engine-driven pumps: one a 10-mgd, horizontal, triple-expansion, condensing, steam-engine-driven duplex pump, installed in 1910; the other a 6-mgd unit.

In recent years, the city of Fall River appreciated the fact that its steam plant was obsolete and that it was expensive to operate, particularly when boilers had to be maintained in service 24 hours a day, year after year, even though the steam pumps themselves were seldom required but they had to be ready for any emergency. Consequently, they made a very complete study of their whole pumping

problem, employing the services of Creer, Kent, Mather, Cruise & Aldrich, engineers from Providence, R. I. The Fall River Water Superintendent, John Moran, represented the city in the negotiations. Mr. Moran has recently retired, after 38 years of faithful service as superintendent, and the job is being carried through to completion by the new incumbent, Maurice Daly, who for many years was assistant to Mr. Moran.

When it was definitely decided that the steam pumps were to be replaced, every conceivable type of driver was given thorough consideration. Although the Fall River Electric Co. has provided very reliable service, it was felt that a second source of power was needed, not only for adequate assurance of standby service, but it should be of such design and type that it could be utilized for continuous operation, should the city of Fall River ever decide to change from purchased electric power. Steam-turbine-driven centrifugal pumps, with steam supplied from a reconditioned boiler plant, were considered but found to be too expensive from the standpoint of operating cost. Gasoline-engine-driven centrifugal pumps proved to be attractive from the standpoint of first cost but were limited to standby duty only. Likewise, high-speed Diesel-engine-driven centrifugal pumps were investigated. They were fairly attractive from a cost standpoint, but here again their use was limited to standby service. After a thorough engineering study was made of all types of drivers, a slow-speed Diesel-engine-driven centrifugal pump was finally selected and purchased, because it not only offered reliable standby service but could also be operated continuously at a low pumping cost.

This is a 12-mgd unit, consisting of a Worthington, model DD7, 4-cycle, 7-cylinder, 13¼-in. bore by 17½-in. stroke, Diesel engine, rated 615 brake horsepower at 360 rpm, driving through a Farrel-Birmingham gear set a Worthington, model 18-LC-3, single-stage, double-suction centrifugal pump. On the engine, the brake mean effective pressure is 80 lb, and the piston speed 1,050 ft per min. This is the standard rating for continuous heavy-duty service and thus provides for the possible future requirement of continuous operation.

Besides the regular domestic water-supply system covered by our discussion, the Fall River Water Department also maintains a

pumping station with a 20-mgd capacity and approximately 4½ miles of 42-in. water main, for supplying water from a separate pond in the adjoining town of Dartmouth to what is known as South Pond in Fall River.

The water in this South Pond is used by a number of manufacturing concerns for industrial purposes only. The Fall River Water Department receives no revenue for services furnished by this 20-mgd-capacity pumping station, but is performing this service because it wants to conserve the domestic water supply in North Watuppa Pond. The Water Department considered this additional service necessary and installed it in 1948, due to the fact that it is required by law to spill water from North Watuppa Pond over into South Pond, should the water level in the latter drop below a certain level.

We mention this point to emphasize the degree to which reliability is stressed in the Fall River Water Department and, at the same time, efficiency is maintained at a sufficiently high level to show a large profit year after year.

The Fall River Water Department is practically 100% metered. At its very beginning in 1875, there were 660 different services on the system, 80 of which were metered. Today there are close to 14,000 services and, for all practical intents and purposes, the system has been 100% metered for the last 20 years. Fall River, like many water departments, realized many years ago that meters meant additional revenue for the department. They have always been progressive and made the matter of the reliability of the system their first concern, as is well known by all who have visited them and observed their facilities.

However, the department has always operated under a very fine business policy, as a result of which it has shown a profit averaging better than $100,000 per year for the last 30 years or so. These profits have made it possible for the department to maintain a very substantial cash surplus and, at the same time, turn over a considerable amount of money to assist in maintaining other facilities in the city outside of the Water Department.

Undoubtedly contributing greatly to the success of the Fall River Water Department has been the fact that the various mayors over the years have appointed outstanding citizens, who have served without pay, as Water Commissioners.

Discussion

Linn H. Enslow (New York, N. Y.). I should like to put Mr. Kane "on the spot." He is general manager of the Columbia Water Co. at Columbia, Pa., and he has a very interesting story—I know that.

Theodore Kane (Columbia, Pa.). About 15 years ago I gave a paper on Diesel engines in water works at the State College, Pennsylvania. I didn't know what I was writing at that time. How in the world I ever got the paper across I will never tell you. Even today, 15 years later, I am by no means an authority on Diesel engines in water works. But I did do this for my own satisfaction: I made a comparative cost sheet of Diesel versus electric power. I did that because just recently we had trouble with one of our Diesel engines. The block split and we had other trouble, and we thought probably we would replace with power. The figures that I worked up and presented to our board of directors were rather startling to them.

I showed that, in the 15 years of operating Diesel engines, we made a saving for the company of roughly $27,000, as against power. Now, Mr. Gallagher, of course you will like the statement I just made, but I am going to make some other statements which you may not like quite as well. With all these savings, if we were to replace our plant completely, I question whether we would use Diesel engines. I think that you will find, with proposals today, in considering power against Diesel, that it is pretty close—that you do not make too much of a saving. When we first put the Diesel engines in, fuel oil was selling at 4 cents a gallon. Today it costs about 10.65 cents a gallon, so that you will see how the cost has gone up.

In regard to maintenance, I figure that, on the average over the 15 years, it would cost about $530 a year for maintenance on a Diesel engine, whereas with power there would be, on an average, only a little better than $100 of maintenance per year. In those figures I include the labor involved in repairing.

Now, there is a lot of trouble with Diesel engines. You can't put a Diesel engine in your plant and let it run, run, run; it just does not work that way. Every hour, we check our exhaust temperatures and our own card readings. We know whether the engine is firing

early or late. We have a lot of maintenance on our Diesel engines. We want them right; in fact, we have to have them right.

One feature of Diesels is that, of course, when they are not operating there is absolutely no cost. If we put in a power standby, there would be a terrific standby charge, and that we could not afford to do. Therefore, we had to repair and had to install a brand new Diesel engine in the plant. Consequently, we have two of them. Of course we shall continue, probably, for the next 10 or 12 years with Diesel engines. I don't know what is going to happen to us with this rise in fuel-oil costs, because our maintenance cost has gone up, as well as our labor cost, etc.

Of course there is one distinction. If someone is putting in a Diesel engine, he will save labor. Then you have another story in favor of power, because, when you put power in, you do not have to have someone watching it all the time. In our case, however, the labor would be the same, because our pumping is right in our filtration plant.

I have shown that operating costs were about $2.23 per thousand gallons with Diesel, as against about $2.93 on power, which gave me my saving. In other words, the cost of fuel for Diesels is about 37 cents per hour, as against a kilowatt-hour of about $1.12. But that figure of 37 cents per hour would probably double when fuel oil doubles. Consequently, if I make another comparative cost setup in the next few years, I may find that we shall not save the $1,800 per year, if that is what the actual saving is in the 15 years. The saving may be cut way down; in fact, the cost may even up. I don't know. But I know that in the next few years I am going to make another study.

That is all I have to say this morning. Thank you very much.

DONALD C. CALDERWOOD (Nashua, N. H.). Thank you very much, Mr. Kane. I was very much interested to hear those remarks, because they practically are parallel to our own experience in Nashua. Back in 1934 we put in a Diesel plant, which many of you have visited. Just a few weeks after it was installed, the president of my company gave a talk before the Association and told about it.

We have a water-power development that furnishes us with approximately 65 to 70% of our annual power requirements. The

balance is made up at the present time by a Diesel. After going through the past years when we had an extended dry spell in New England, we found that our water power was depleted and that the Diesel was our chief source of pumping. We felt that we should at least have another Diesel or some other means of pumping water —sort of an extra string to our fiddle, as it were. The old steam plant that this Diesel replaced was the second string on the fiddle. It was more or less antiquated, had not been run for several years, and we knew it was of no further value.

When we first put in the Diesel, the entire Diesel plant cost us approximately $20,000—a 250-hp Diesel with all fixings, all installed and ready to go. The only things we had to furnish were the foundations and the buildings. The fuel oil that we purchased at that time cost us 4½ cents a gallon; lubricating oil was about 40 cents a gallon, and other costs were comparable. The last oil we bought was 10.6 cents. Lubricating oil has gone up—not in the same proportion but it has gone up somewhat. Repairing and maintenance costs have gone up considerably. We figure that today it would cost us approximately $65,000 to put in a good Diesel plant.

For our new installation we have on order with the Worthington Pump & Machinery Corp., at this moment, two electrically-driven pumps. The power rate is quite favorable, and we were able to work out a scheme with the local power company whereby, for an annual guaranty which we feel we can use up very easily in power, the demand charge will be absorbed by the power used. We feel that we can install the complete electrically-driven plant for about $25,000, which would include the connections and everything that goes along with it. On that basis, when you consider the overhead and annual carrying cost, it far overbalances any small fuel or power savings you would get in having a Diesel plant. We still have our original Diesel and we like it. We think it is a good source of power. It is one of those cases where, when a change of economy takes place, we can change from one source of power to another. Also, there are many factors which enter into the picture, such as reliability and the continuity of the service that a water utility has to give to its customers.

Are there any further comments to be made on the paper?

MR. ENSLOW. I should like to ask this: Do you think that the

fact that you had a good Diesel operating and could buy another one, and so on, had anything to do with this favorable power rate that you have reported? That is to say, what influence do you think that the Diesel availability—Diesel "go-ability"—had on this power rate that you have received?

MR. CALDERWOOD. I don't know that the Diesel had too much to do with the actual power rate. In fact, it is not really a special power rate that they have given us, but it is taking advantage of certain conditions in the regulations and the rate as published for anyone, which are not usually invoked or used—let me put it that way. It is a special way of figuring the rate, where you have a power of your own that is available. There is no question but what there is a willingness on the part of most power companies to sharpen their pencils, if they figure they are up against some competition.

In our particular case, we are putting in two electrically-driven units, which will give us more flexibility. Except for possibly ten days in the year, one electric unit, operating on a 24-hour schedule, would supply the needs of the city. I believe the whole problem is one that has to be determined on individual merits and conditions, rather than by any general formula that allows you to say, "Well, you may do this or do that."

H. H. POTTER (Augusta, Me.): I had hoped somebody would ask, or state, about what the cost per kilowatt-hour for electricity would have to be, before the Diesel plant would be more expensive.

MR. CALDERWOOD. Possibly Mr. Gallagher can give us some figures on that basis.

MR. GALLAGHER. I am fortunate this morning in having Mr. Boehm of our company with us. He wants to make some comments on Mr. Calderwood's remarks and he will also answer that question.

A. M. BOEHM (New York, N. Y.). This matter of fuel-oil costs has us concerned as well as you gentlemen. The problems of any company that is engaged in selling Diesel engines increase as the fuel-oil costs go up.

I was very much interested in Mr. Kane's statement, because it reflects pretty much what we hear all around the country. It is becoming more and more difficult to justify a Diesel generating unit as against purchased electricity. But as against that you have rather

an opposite story, where every Diesel-engine builder was never busier in his life than he is right now. Where they are going is a matter of competition with electric rates.

The fuel-oil cost, while it is an important matter, is not the only offender in that regard, I might say. The cost of purchased electricity might also be mentioned. I should like to make the point that the purchased electric rates that you find in the schedules may not necessarily be firm rates, because, in the fine print of almost every purchased electric rate—in the contract—you will find a coal clause, where the rate that you are charged is on a sliding scale, depending on the cost of fuel to the power company, to make the kilowatt-hours that they expect to make from their steam plants. We have had instances where that has accounted for a very considerable increase in the overall rate. Yet the basic rate printed in the schedule remains the same.

With fuel oil at, say, 14 cents a gallon, which some parts of the country are paying, results can be obtained with a so-called supercharged Diesel, where not only the rate at full load is low but the average consumption over a wide range of loads remains almost constant. In that type of engine you will easily turn out 14 kwh to the gallon. For fuel oil it becomes a matter of dividing the cost per gallon by 14 kwh. If you are paying 14 cents per gallon, it means that the average cost is only one cent; at 11 cents, which you are paying, it is 11/14 cent. In other words, it is ¾ to 1 cent per kilowatt-hour. With lubricating oil, maintenance, labor and fixed charges, you are lucky to get out under 1¾ cents per kilowatt-hour.

A MEMBER. I do not pretend to be an expert on Diesel fuels, but I have heard occasionally of some operators using a crude form of oil, filtering it themselves and getting their oil at a much lower cost. I wonder if that is a factor that anyone could throw some light on for general practice.

MR. BOEHM. Our experience in that regard has been that there is possibly a maximum saving of 3 or 4 cents a gallon, but it involves additional equipment and additional first cost. The engine can take it all right, provided you do certain things to it ahead of time, knowing that you have to burn this heavy oil. You have to centrifuge it and you have to heat it, so that you maintain viscosities which are

comparable, say, to your No. 2 or No. 3 fuel oil. Usually the heavier oils have a higher sulphur content, and your fuel pumps and spray valves have to be of special stainless-steel material. But it can be done. We have an order now for 42 engines for the Trans-Arabian pipe line, where they are going to burn the crude right out of the line, as it comes from the field. There will be six stations along the line, and they will tap into the crude line and use this oil in the engines. Thus it can be done, if you are willing to pay a little more in first cost and if your operators are willing to put a little more time in on it, because the centrifuging requires cleaning. It is usually dirty.

THE MEMBER. That is a purchase price of 3 or 4 cents a gallon, against 10 cents?

MR. BOEHM. That is right.

MR. CALDERWOOD. Of course, there is the availability factor which wants to be considered. Up here in New England, where most of us come from, this No. 2 household fuel, or even in some places the Diesel fuel, is readily available. For the most part, the dealers and distributors do not have this heavier fuel on hand. It would take special arrangements to get that and you would probably have to use very large quantities of it, in order to have the saving amount to very much.

PAUL F. HOWARD (Boston, Mass.). I should like to point out that in purchasing fuel oils one should be careful to take into consideration their weight. In connection with the plant at Burlington, Vt., we received bids for furnishing Diesel oils and found about a 10% variation in their weight. It so happened that one of the highest bidders was actually the lowest bidder on a gallon basis. But when you took into account the weight of it and the power you would get out of that oil, it was one of the higher bidders that had the best proposition for the city.

That plant at Burlington is interesting in several respects, along the lines that have been discussed. One is that the city of Burlington has two power stations—that is, one is owned by a private power company and one is owned by the city—and the poles are on different sides of the street. You can subscribe to either city-owned power or privately-owned power and, when you get mad, you shift over and shift back, but it averages up about the same.

Now, the Burlington Water Department put in a Diesel engine—as I recall it, a 285-hp engine—really at the request of the city-owned light department, known as the Power Department. The latter department said, "Look here, if you get off our lines—we sell it to you at the wholesale rate—we can use the same capacity, sell at retail and make some money for the city." Therefore, the water department did put in a plant and it actually paid off the investment in about four years.

The situation there is rather interesting. Burlington gets its water from Lake Champlain and it is filtered. It goes from the main-line pumps from the clear well into the distribution system. There is one other saving which Burlington makes. They use the exhaust heat for preheating the return water through the heating plant. They have one central heating plant, that heats the filter plant, the garage, the pump room, the work shop and the storage room. They run the Diesel engine in the wintertime perhaps 20 to 22 hours a day and use the exhaust to preheat the water, with the result that their saving last winter was 20 to 25% of their fuel bill, due to the recovered heat from the exhaust.

MR. BOEHM. I should like to interject one other thought. In Midland, Mich., they have done something that I have never heard of before or since. In addition to utilizing the heat of both the jacket water and the exhaust, they then bubble the exhaust gas through the city water supply, to break up the CO_2 content.

MR. CALDERWOOD. This is the first time I ever heard of a Diesel as a chemical treatment plant.

HORACE L. CLARK (Sanford, Me.). I should like to ask Mr. Gallagher to say something about engine speeds that he recommends these days for water-works plants. I know they vary all the way from 320 up to 1,200 or so.

MR. GALLAGHER. Why, 360 was the one they were putting in at Fall River. Around 450. I shall let Al Boehm go a little farther on that.

MR. BOEHM. We are pretty much of a slow-speed outfit, as far as Diesels are concerned. The highest-speed engine we build now is 600 rpm. We are firmly convinced that, where you want to run the engine continuously day in and day out, it is still pretty good sense

to stick to speeds around—I am not referring to revolutions per minute but piston speeds—certainly not more than 1,200 ft a minute. We have built engines up as far as 1,400 ft a minute, 600 rpm. That is our story. If you talk to some other engine builders, you will get an entirely different story, because they may have engines that go up to 1,200 rpm.

DIESEL PUMPING UNIT FOR MANCHESTER WATER WORKS

BY CLARENCE L. AHLGREN*

[*Read November 18, 1948.*]

DIESEL drive for pumping is receiving more attention in New England. This matter of increased interest brings up the question of "why", which rapidly takes on the aspects of a major survey. However, we can examine the reasons for the Manchester installation, which was completed early in January, 1948. For this examination we need a word picture of the general development of the pumping plants there.

Manchester straddles the Merrimack River about 20 miles north of the Massachusetts state line. Generally, it spreads out flat near the river banks and to the southeast and southwest, and rises in elevation to the northeast and northwest. Its development was the normal of most cities—the industries crowded the river bank, while the people built their homes on the lower elevations, until the more enterprising, who wanted a view of the city, pushed to the height of the land.

Around 1870 a demand was made for a municipal water supply. It was natural for the engineers who were retained to turn to Lake Massabesic, which lies roughly parallel to the Merrimack River and 3½ miles east of it. Here was a body of water with a capacity of 15 billion gallons, which had a surface area of 4 sq mi and a watershed of 40 sq mi. There should be enough water for consumption and enough to generate power to pump it. At the outlet of the lake the engineers built a station to do just that, a water-wheel being direct-connected to a pump. An 18-mil gal reservoir was built on a rise in the land between the pumping station and the river, and the distribution system laid out. The "industry" along the river—the great Amoskeag Manufacturing Co.—used only a very little Massa-

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besic water, mainly for drinking. The great bulk of their use—for fire protection, for process water, for waste disposal, etc.—came from their own 11-mil gal river-water reservoir, which was built half-way up the northeast hill. This use of river water was an important factor in the volume of municipal consumption then and was to become an important factor again just before World War II.

By 1893 the push of the population toward the higher northeast section of the city was sufficient to warrant a reservoir at a higher elevation than the original one. The engineers decided that a second pumping station on the lake, as far north from the outlet as permissible, should be built to supply this reservoir, which for some reason was held to a capacity of 4 mil gal. Thus, a second system, independent of the original and higher in pressure, was established. The independence included power for pumping. As it was decided to use steam, a boiler room and pump room were constructed, to house two 3-mgd horizontal-acting steam pumps.

By 1900 consumption on this system had increased sufficiently to require a 6-mgd unit. An addition to the station was built and a Snow-Holly, triple-expansion, vertical steam pump was installed.

Thus Manchester had two pumping stations with prime sources of power directly connected to pumps. However, they also had dry years then. Around 1910, a dry year indicated that an additional source of power was necessary at the low-pressure station and, consequently, motor-driven centrifugal pumps came into the picture. A 6-mgd unit was purchased and installed, and a power line was run to the station by the Public Service Co. The installation was a success. The unit was simple to operate and required a minimum of floor space and manpower. By 1915 it was decided to replace the direct-drive water-wheel pump with a water-wheel turning a generator. This would furnish power to motor-driven units at the low-pressure station and, with a power line, to any units that might be installed at the high-pressure station. Along with the generator a second, motor-driven, 8-mgd centrifugal pump was installed.

The idea of having two sources of power available, as well as the simplicity of motor-driven units, was here to stay. In 1917 the original steam units at the high-pressure station were replaced by two motor-driven units, one 5-mgd and one 3-mgd. The larger of these

was limited in size by the capacity of the generator at the low-pressure station. In addition a 1,000-kw steam turbo-generator was installed. The only justification that we can figure for this unit was that it perhaps kept purchased power rates in line. The Snow-Holly triple-expansion unit remained as an important unit for summer pumping.

Then, from 1917 to 1933 no major changes were made at either the high-pressure or the low-pressure station. During that time numerous proposals were considered for improving steam-generating conditions and for installing Diesel drive in place of the steam. However, since the capacity of the units and the flexibility of operation were sufficient to meet all conditions of consumption, the proposals were not favored.

In 1933 Percy A. Shaw was appointed Superintendent and Engineer. His first job was to make a survey of the plant he was to direct. The survey revealed the following and indicated the plan of attack:

1. Consumption in Manchester had not increased during the past 15 years. The potential increase appeared to be within the capacity of the pumping plants.

2. Water for power was being used about 50% of the year. There were many possibilities of improving the efficiency of this use.

3. The 4-mil gal capacity of the high-pressure reservoir was dangerously out of line. A 17-mil gal unit was constructed in 1934 to correct this.

4. The water-wheel driving the generator at the low-pressure station could be rebuilt, to increase the efficiency about 15%. This was done in 1936. A double horizontal wheel with individual gates was installed. One wheel generates enough power for operating the largest pump at the low-pressure station. Two wheels generate enough power for operating the largest pump at the high-pressure station. This limit of capacity at the latter station was to influence our choice for a Diesel drive in 1946.

5. The efficiency of the pumps and piping in the low-pressure station could be improved. In 1937 a new 8-mgd pump and improved hydraulic design of piping gave us a saving of 15% on our power bill.

6. Spring floods were sending valuable water power over the

lake outlet spillway each spring. In 1938 a 1.3-billion gallon storage reservoir at Tower Hill, one of the main tributaries, was constructed to conserve this spring waste and to improve quality. This reservoir is filled each spring and emptied in the fall. It may be used for consumption during dry years or for power during wet years.

7. The efficiency of the centrifugal pumps and piping at the high-pressure station could be greatly improved. The 1,000-kw steam turbo-generator lacked boiler capacity and efficiency. In 1940 the 1,000-kw unit was sold. A new, 7-mgd single-stage pump was installed, with vastly improved piping hydraulics. The increase in efficiency allowed us to gain in capacity from 5 to 7 mgd and yet keep within the limits of the generator at the low-pressure station.

8. High-pressure water could be let into the low-pressure system, but to get low-pressure water into the high-pressure system required a booster station. Such a station was constructed in 1941. The idea was stimulated by the pending participation in World War II.

9. The high-pressure system was the key system, since its supply could be used in both systems. Therefore, its standby unit should have capacity and reliability. By 1942 the Snow-Holly unit and the steam-generating boilers were 40 years old, required a large crew of trained men, and lacked capacity. However, World War II had engulfed us before details were worked out for its replacement. For four years the planning for this operation smoldered along. By 1946 it flamed out, along with the rush of post-war planning.

Thus, when studies were resumed in 1946 on how to strengthen the independence of the high-pressure pumping plant, the facts lined up as follows:

1. *Consumption.* Here is what happened to the hitherto static consumption pattern from 1934 to 1946. In 1934 the daily average for Manchester was 5.25 mgd. By 1938 it was down to 5.08 mgd. A war on waste and a program for improved meter repairing and metering had lowered slightly a static daily average. Then things began to happen.

The great mill properties of the Amoskeag Manufacturing Co. were being sold off individually. No concern wanted to operate the river-water system. The proposal was made and carried that it be

turned over to the Manchester Water Works. Since the Department was not in the river-water business, it was decided to abandon the reservoir and the mains were prepared for Massabesic water. The daily average consumption rose to 6.2 mg in 1939—an increase of 1.2 mgd. In 1941 Grenier Field Airbase requested water. This service, plus increased industrial use, added another mgd to the average. By the end of 1944 the average was up to 7.53 mgd, with peaks of 12 mgd in the summer months. Potential increases were:

(a) Some industrial use;
(b) An unknown quantity of air-conditioning use.

It is interesting to note here that increased domestic consumption was not considered too great a factor, but with a huge building program, higher wages and freer spending, there has been a notable increase.

2. *Pumping Plants.*

(a) The pumping plants were adequate for prevailing daily average consumption, with the exception of the weak stand-by condition at the high-pressure station.
(b) There was room for increased average consumption.
(c) Summer peak loads might exceed in the near future 24-hour operation of the largest electric unit at each station. When this became a reality, a larger electric unit might be installed at the high-pressure station. We pointed out that at present the size there is limited by the water-wheel capacity. If a larger electric unit were installed, water power would then be limited to use at the low-pressure station.
(d) When the daily consumption average exceeded 7.5 mgd, purchased power must at all times be used to supplement our own water power. Since we establish our demand charge for purchased power each month, this would mean a high cost for a small power use.
(e) Capacity of pumping mains, intake and Venturi meter must be considered.
(f) Available space in the Snow steam-pump room was a factor.

3. *Hydrology of Our Watershed.* Lake Massabesic is a one-year-storage proposition. Each year it must be used to capacity, to provide storage for the next spring run-off. This means that power will be available for motor-driven centrifugal units for some time.

It was evident from the facts that we needed a stand-by unit to:

1. Supply peak demands in the summer;
2. Put off into the future as far as possible the time when a larger motor-driven unit would have to be installed at the high-pressure station;

3. Be used once each week now and in increasing amount in the future, to supplement water power and thus keep purchased-power demand charges to a minimum.

When all the facts were charted and properly balanced, they indicated a direct-drive Diesel pumping unit.

Specifications and Installation Plans

Work on the specifications was begun immediately. In May of 1946 bids were called. A careful study of the bids resulted in the selection of a 960-hp, Fairbanks-Morse, opposed-piston, model 38 D, 8⅛, 720-rpm Diesel, direct-connected to two single-stage pumps in series, having a 12-mgd capacity. Delivery was to be the following year. Work on installation plans was begun. The following points were developed from the specifications. (In order to save listing the same points again, we shall make operational comments on them as they are taken up.)

1. *Intake Air.* Intake air is drawn through flat, cell-type filter frames, set vertically in a brick filter-house, outside of the pump room. An intake silencer is mounted on top of the filter house. Before raising the intake pipes into place, they were thoroughly cleaned of mill scale on the inside and painted with a rust-inhibiting paint.

2. *Exhaust.* Exhaust is through a side-inlet Maxim Spark Arrestor-Silencer, mounted on the filter house. We have found the Spark Arrestor also keeps small particles of carbon from being blown out upon our grounds. After installation and trial runs, we found it necessary to add another wave trap close to our engine, because of the 35-foot-long exhaust line. The line is designed to keep all expansion strain off the engine.

3. *Fuel System.* Our 10,000-gal storage tank is 400 ft from the Diesel room, back out of sight from the building, on a rise in the land. Fuel flows by gravity through a 2-inch pipe down to the building, inside, through a meter, and outside again to a 300-gal day tank in a concrete pit. From there it is picked up by the Diesel fuel pump and pushed through a Nugent filter to the Fairbanks injection pumps. An inspection of the tanks last month revealed no

sludge or water to date. The Nugent fuel filter does pick up a slime deposit slowly. During the first 500 hours of operation, nozzle sticking was encountered. For the next 500 hours a lubricant was added to the fuel at the day tank, which cleared the sticking. There has been no nozzle trouble for the next 1,000 hours.

4. *Lubricating-Oil System.*

(a) *Cooling.* The lube oil is picked up from the engine sump by a pump, gear-driven from the engine, forced through a Ross heat exchanger and a full-flow edge-type filter into the engine. The oil is bypassed around the exchanger by a Sylphon valve, in the correct proportion to attain the correct temperature of the oil leaving the engine. This temperature was set at 145° initially. We have recently raised it to 165° in conjunction with lubricating-oil studies. We run about a 7° differential. Raw water is used to extract the heat in the cooler. It flows through at a constant rate.

(b) *Filtration.* Based on an estimated running time of 1,000 hours per year, we chose a bypass type of filter, that would take either fuller's earth or cellulose cartridges.

(c) *General.* This year we will run 2,500 hours, in order to become acquainted with the unit and to further our lubrication studies. With the temperature at 145° our oil pressure ran 33 lb per sq in.; at 165° it is 27 lb per sq in.

(d) *Pre-Lubrication.* If the unit is down for any length of time, we have a small pre-lubricating pump that can be started up, to push oil into the engine before starting.

5. *Cooling-Water System.*

(a) We employ a closed system for cooling water. A gear-driven circulating pump forces water through the engine, out, through a Ross heat-exchanger and back to the engine. The circulating water is bypassed around the cooler by a Sylphon valve, in the correct proportion to obtain a temperature out of the Diesel engine of 150°. Raw water that has already been through the lube-oil cooler flows at a constant rate through this circulating-water cooler, to pick up the heat.

(b) A standby circulating pump for the closed system is used

to extend the cooling-down period each time the Diesel is shut down. This prevents metal-contraction strains. We run it for about 25 minutes.

(c) For circulating-water treatment we are using, with good results to date, a metallic insert, much the same as is used in hot-water tanks to prevent scaling and corrosion.

6. *Raw Water for the Heat Exchangers.* We pipe water from our force main through a pressure-reducing valve to the exchanger. It is controlled to a minimum of use by hand-operated valves that take advantage of seasonal temperatures. The standard installation of a raw-water pump, direct-connected to the engine, was eliminated by the extreme suction lift encountered.

7. *Starting Air.* We have starting-air capacity for eight normal starts. The three 20-in.-by-60-in. tanks are filled to 250 lb per sq in. by a Curtis air compressor with automatic controls, driven by a Fairbanks-Morse motor. For emergencies a gasoline drive is available.

8. *Gauge Board and Alarms.* On the Marshalltown gauge board we required:

(a) Pyrometer readings on individual cylinders and on combined exhaust;
(b) Pressure and temperature of lube oil;
(c) Temperature of circulating water;
(d) Pressure of raw water;
(e) Pressure of starting air;
(f) Scavenging air pressure.

For alarms we required a battery-operated circuit for oil pressure and temperature, and for circulating-water temperature. We also have additional thermometers on "in" and "out" of lubricating oil, circulating water and raw water, and additional gauges on raw-water pressures, lube-oil pressures, starting-air tank pressures, and pressure drops across fuel-oil and lube-oil filters and air filters.

9. *Pumps.* Usually, when a water department goes out to purchase a pumping unit, they spend most of their time on pump specifications. In the case of Diesel power as the driving unit, more time is spent on specifications for this unit. However, we did go into the

pump specifications thoroughly. The tests of the pumps were witnessed at Beloit by our engineers and all specifications were met or exceeded. Field tests after installation checked the factory tests. The two pumps are single-stage, bottom-suction units, connected in series. The priming system is a simple Skidmore water ejector that we use on our other pumps. Gauges indicate vacuum on the suction and pressure on the discharge. A Venturi meter measures the pumping. The pumps discharge through the standard gate-valve-check-valve combination, with one exception—the check valve is 40 ft from the pump.

Construction and Installation

Construction and installation began during the summer of 1947. It is the policy of the Department to do all our own work. A small crew proceeded slowly, guided by an occasional check by Fairbanks-Morse installation engineers. Our biggest problem was lining up the pumps after the right series piping was bolted into place. We discovered that air temperature had a pronounced effect on our work and we were forced to compromise on a mean-temperature alignment.

In order to guarantee a minimum of flushing of the cooling system for the Diesel, we hand-brushed and cleaned every piece of pipe used in connecting up the heat exchangers before installing it. We also dismantled the coolers and thoroughly cleaned them. We found this paid dividends when starting up. By January 2 of 1948 we were ready to make a trial run on the unit. Fairbanks-Morse sent an engineer from their plant to start the unit and make what adjustments were necessary. The adjustments were of a minor nature. A field test was run on the unit for a comparison with factory-test figures. The check was satisfactory and the unit was placed in service. It has operated approximately 2,000 hours since.

Operation Experience. Our operation experience is, of course, limited and the important points have been stressed under Specifications and Installation Plans. The general picture may be more complete by reporting that:

1. The operation time has varied from 8 hours to 20 hours per day.
2. The first two complete changes of lube oil were made after 550 hours of operation. The third change is due soon.

3. Filter cartridges were changed when the first two oil changes were made. They were changed at the 500-hour mark during the present run.

Costs of Operation

The costs of operation are listed in the following table. They are worked out on the basis of cost per kwh, for comparison with purchased power. It is pointed out that they are preliminary, in view of the comparatively short time we have been operating.

Costs of Operation of Diesel Pumping Unit

Item	Gal per mg	Gal per bhph	Gal per kwh	Unit cost of item	Cost per kwh (mills)
Fuel	83.6	0.0572	0.0765	11.037¢	8.45
Lube oil, make-up	0.83	0.00057	0.00076	84¢	0.638
Lube oil, changes	0.59	0.0004	0.00054	84¢	0.455
Lube oil, total	1.42	0.00097	0.00130	84¢	1.093
Filter cartridges				$40 per change	0.120
Water treatment				$25 per year	Neg.
Auxiliary operation				$20 per year	Neg.
Total					9.663

The total indicates an operating cost of 9.66 mills. We have not had any replacement and maintenance costs to date.

Our estimate of what we might expect will bring this figure up to less than 1 cent per kwh, if we average 2,000 hours of running time per year. If depreciation is charged on this same basis, the kwh cost will be in the vicinity of 1.3 cents.

Labor costs have not been included. Our normal complement has taken the operation of the unit in stride. Other costs of heat, light, etc., are normal to the operation of the motor-driven pumps also.

Conclusion

In conclusion, it is evident from the foregoing that our interest in Diesel drive was based on:

1. Development of the engineering approach of the past;
2. Independence of operation;
3. Flexibility;
4. Reliability.

It should also be noted that our daily average consumption has increased ½ mgd each year for 1947 and 1948. This increase has emphasized that our selection of a Diesel drive and of the size of the unit was sound.

INTRODUCTION OF PRE-CHLORINATION OF SLOW SAND FILTERS

BY SAMUEL JACOBSON* AND MARSHALL S. WELLINGTON**

[*Read September 16, 1948.*]

THE Whitney Filter Plant of the New Haven Water Co. was placed in operation in March, 1905. It filters water from Lake Whitney, a distributing reservoir having a capacity of 258 mil gal and a watershed area of 37.7 sq mi that is densely populated. Only the part that is close to the reservoir has a sewerage system. The raw water has an average pH of 7.3; the alkalinity averages 33 ppm and the hardness 49 ppm, both as calcium carbonate; the turbidity averages 8 ppm and the color 30.

The filtration plant is of the slow sand type and consists of twelve beds, each one-third of an acre in area. Six beds are on each side of the operating corridor. Details of design and construction have been reported by Ferry (1) and (2). The beds are cleaned by scraping about one-half inch of sand from the surface. More sand is removed if a run has been extended by raking, a rare occurrence at this plant. The dirty sand is washed hydraulically in an Allen Hazen washing machine and then transferred to a bed whose sand depth is being built up from a minimum of 30 in. to approximately 44 in., during which time it is kept out of service and designated as the "sand bed". *The lower levels of sand and gravel have not been cleaned since the plant was first placed in operation.*

The water enters each bed by flowing upwards through a five-foot-square concrete well, the top of which is a few inches above the level of the sand. The depth of water in the filter is maintained by a float valve in this well. The filtered water flows through a pipe equipped with a Venturi tube and meter. There are connections to a cross-over pipe, by which any bed can be backfilled with filtered water, and also

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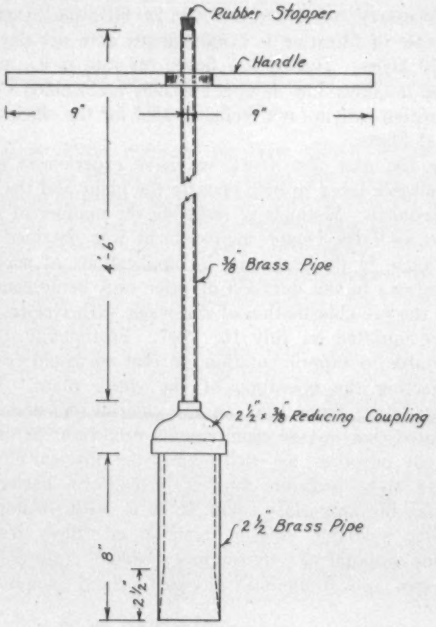
open connections to the sewer line. Chlorination of the effluent from the plant has been practiced since January 28, 1919.

A 750,000-gal clear well receives the plant effluent. This capacity is insufficient to meet the present variations in pumpage, which makes it necessary to reduce the rate of filtration at night. Normally, the rate of filtration is 2 mil gal per acre per day (m.g.a.d.) for about 20 hours. During low flows, the rate is 0.6 m.g.a.d. On Sundays and holidays, low flows are usually maintained. Such variations in filtration rate are not recommended for the efficient operation of slow sand filters.

During the past few years, we have experienced difficulty in obtaining suitable labor to help operate the plant and the labor turnover was frequent. Methods of reducing the number of filter scrapings were considered. Some improvement was obtained by careful control of algae in the reservoir, by applications of copper sulfate. Greater increases in the duration of filter runs being sought, experiments with the pre-chlorination of the water with free-residual chlorination were initiated on July 10, 1947. Fortunately, our plant is easily adaptable to experimentation, so that we could treat one bed without affecting the operation of the entire plant. Preliminary observations dating from 1943 on the chlorine demand of the raw water indicated that dosage requirements would not be excessive.

For study purposes, we used mainly the physical characteristics of the water, most probable number of coliform bacteria per 100 ml of sample, the agar-plate count at 20°C. with 48 hours' incubation, the free available chlorine residual or "flash residual", the total chlorine residual or "five-minute residual", the chlorides, ammonia nitrogen and albuminoid nitrogen. Sand samples were also examined.

To obtain intact samples of sand while the filter was in operation, a sampler was made as shown in Figure 1. The cutting cylinder, an 8-inch length of 2½-inch brass pipe, was beveled at one end to a cutting edge. The other end was threaded and connected with a 2½-inch-by-⅜-inch reducing coupling. A 4½-foot length of ⅜-inch brass pipe was threaded into this fitting. About 2 in. from the other end of the pipe a cross-piece was attached to act as a handle. All joints were coated with lead paint and made air-tight.



SAND SAMPLER

FIG. 1

The following procedure was used in collecting sand samples. A rubber stopper (No. 00) was tightly inserted into the pipe. The cutting cylinder was lowered into the water to the surface of the sand and the cutting edge forced 4 to 6 in. into the sand, with a turning motion. The sand sampler was then slowly and vertically withdrawn from the water and set over a piece of cardboard. The stopper was removed and the sampler was slowly raised. Occasionally, the cutting cylinder had to be gently tapped before the plug of sand would slide out (see Figure 2).

Comparison of Pre-chlorinated and Normally Operated Filters

A main-sterilizing chlorinator was set up on the floor of the operating gallery, with the chlorine-solution hose discharging into the bottom of the influent well at one end of the newly scraped Bed

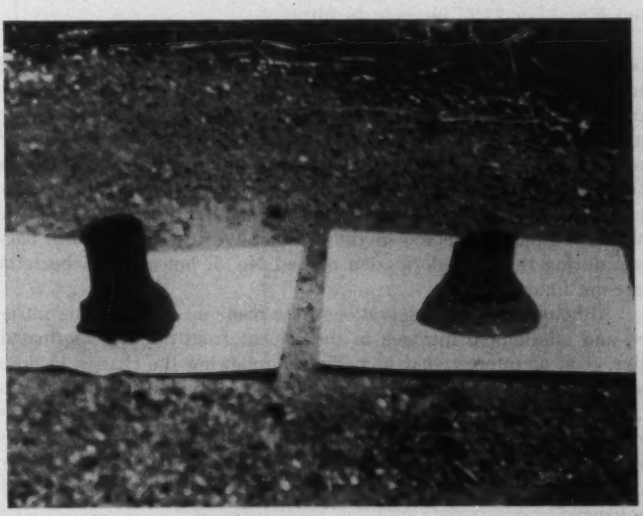


FIG. 2.—PHOTOGRAPH OF SAND SAMPLES.
(Dark sample after normal operation, and lighter-colored sample after pre-chlorination. An irregular fissure in latter sample resulted from handling.)

No. 3. A chlorine dosage rate of 2 ppm was used at the start. This was gradually increased as shown in Figure 3 so that, at the end of the run, the chlorine dosage rate was 6 ppm. Bed No. 4, located opposite Bed No. 3, was scraped and placed in operation at about the same time and at the same filtration rates, so that a comparison could be made.

Fig. 3 indicates the different rates at which the loss of head developed in both beds and shows that it increased in Bed No. 4 about twice as fast as in Bed No. 3. The data also show that, when the chlorine dosage rate was increased, the rate of loss of head was retarded. At 6 ppm chlorine, the loss of head dropped from 21 in. to 14 in., followed by a precipitous loss of head paralleling that of the normal filter. The total quantity of water filtered by Bed No. 4 was 89 mil gal per acre (m.g.a.) as compared with 153 m.g.a. for Bed No. 3—a 72% increase in filtration.

Residual chlorine was not detected in the effluent at any time. Residuals observed in the influent well are given in Table 1 and indicate free-residual chlorination at about 3 ppm chlorine, at which dosage rate very little combined available chlorine was present. The variations in the bacterial counts and M.P.N.'s of the raw water indicated changes as affected mainly by run-off. The variations in the bacterial counts and M.P.N.'s of the effluent water were mainly dependent on chlorine dosage rates. Lactose broth tubes showing gas were usually observed in the presumptive test for coliform bacteria during the pre-chlorination of Bed No. 3, but coliform bacteria were not found.

Although too few observations were made on the ammonia nitrogen and albuminoid nitrogen in the effluent from the pre-chlorinated filter, it is of interest that immediately following the change in dose from 4 ppm to 6 ppm on the 55th day, the ammonia nitrogen was 0.126 ppm. The effluent from the control filter had 0.032 ppm ammonia nitrogen. With continuing chlorination, the ammonia nitrogen decreased so that 16 days later it was observed to be about the same as that from the control filter. The albuminoid nitrogen remained unchanged.

Table 1 shows that, except for a few moderately high bacterial counts at the beginning, a maximum count of 420 per ml at 20°C. was observed on the 55th day, coincident with the high ammonia nitrogen.

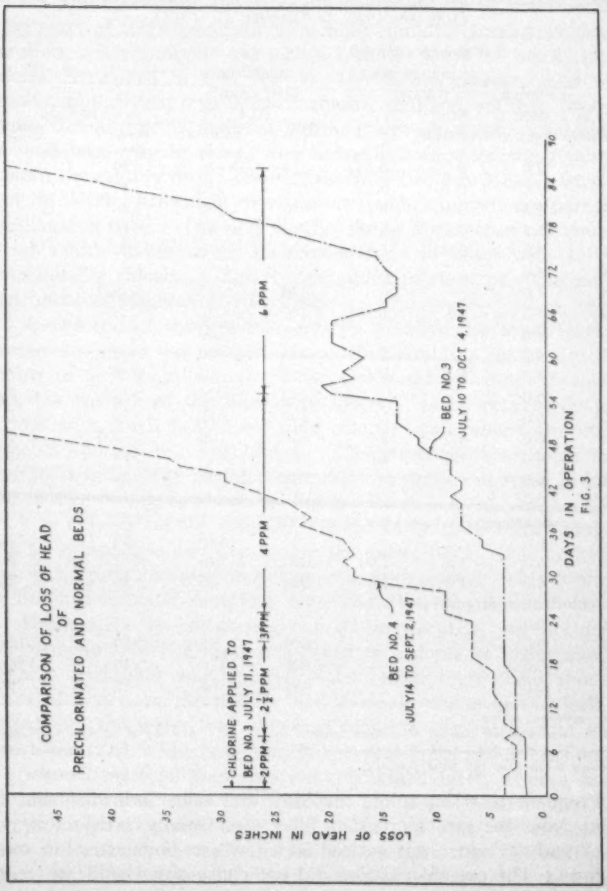

COMPARISON OF LOSS OF HEAD
OF
PRECHLORINATED AND NORMAL BEDS
CHLORINE APPLIED TO
BED NO.3 JULY 11, 1947
2PPM
2½ PPM
3PPM
4 PPM
6 PPM
BED NO. 4
JULY 14 TO SEPT 2, 1947
BED NO.3
JULY 10 TO OCT 4, 1947
LOSS OF HEAD IN INCHES
0
5
10
15
20
25
30
35
40
45
0
6
12
18
24
30
36
42
48
54
60
66
72
78
84
90
DAYS IN OPERATION

FIG. 3

TABLE 1.—EFFECT OF PRE-CHLORINATION ON BACTERIOLOGY OF BED NO. 3.
(July 10, 1947, to October 31, 1947)

Days in operation	Chlorine dose (ppm)	Chlorine residual at influent well (ppm) Free available	Total	Standard agar-plate count, 20°C. Raw	Effluent	M.P.N. Raw	Effluent
1	2.0	0.05	0.60	350	17	39	
7	2.0	0.10	0.25	170	350	39	
8	2.5	0.10	0.25	640	180	>39	
20	2.5	0.25	0.80	760	40	15	
21	3.0	0.90	1.1	920	33	39	
24	3.0						
25	4.0			230	27	< 2.0	
29	4.0	1.4	1.5	140	32	8.9	
33	4.0			140	200	< 2.0	< 2.2
36	4.0	2.0	2.0	330	50	2.2	
48	4.0	1.2	1.4	130	12	>39	< 2.2
55	6.0			170	420	>39	< 2.2
56	6.0			190	400	>39	< 2.2
64	6.0	2.0	2.0	310	56	< 2.0	< 2.2
77	6.0	1.0	2.0	80	15	>39	< 2.2
78	6.0			92	15	39	< 2.2
86	6.0						
	—Raked—						
87	6.0			75	120	39	< 2.2
95	6.0	1.8	2.0	1,000	45	< 2.0	< 2.2
110	6.0	2.0	2.0	1,000	30	>39	< 2.2
—Pre-chlorination Stopped—							
111	0.0						
113	0.0			650	35	>39	16.1
129	0.0			2,300	170	>39	>16.1
136	0.0			480	68	>39	9.2

ppm—parts per million.

M.P.N.—Most Probable Number of coliform bacteria per 100 ml. A value of <2.0 indicates absence of coliform in a 1-ml portion and in all 5 of the 10-ml portions. A value of <2.2 represents absence of coliforms in all 5 of the 10-ml portions.

Frequent tests on color, turbidity and odor indicated that the effluent from the pre-chlorinated filter was usually satisfactory. A slight "mud-flat" odor was noticed on a few occasions after low week-end flows. The pre-chlorination did not cause any significant reduction of color in the effluent.

Sand samples from the same places in Bed No. 3 and Bed No. 4 at the start of the experiment were both similar. During 49 days of operation, a layer about one-eighth inch in depth of black, fibrous material developed in Bed No. 4. Under the microscope at 100X magnification, living organisms, including diatoms, rotifers, protozoa, minute thread worms and flat worms, were observed. Underneath, the sand was gray in color, with a small amount of slimy material between the sand grains. The sample from Bed No. 3 had a very thin layer of dark, brownish, non-fibrous material over comparatively clean-looking sand. The only motile, living forms observed were the thread worms living under these conditions in water with 1.4 ppm free available chlorine. Fig. 2 is intended to show how the samples from both beds looked at this time.

At the end of the run for Bed No. 3, when the water level was lowered to about six inches above the sand, an unusually large number of bubbles appeared. When the water had drained from the sand, the surface of the filter was observed to be very badly pock-marked with small craters of light-colored sand, about one inch in diameter and about a foot apart. The presence of so much air in the filter was in large part the cause of the sudden increase in loss of head in Bed No. 3, as shown in Fig. 3.

The deposit on the sand was found to be dark brown and uniform in appearance and depth over the entire filter. The underlying sand was lighter in color and cleaner in appearance. This confirmed our findings by sand sampling while the filter was in use.

Because of its surprisingly good appearance, we decided to rake Bed No. 3. The filter was raked to a depth of about one inch and was backfilled with filtered water, until there were about six inches of water on the sand. The drain to waste in the well was opened and the water was skimmed off at a low rate, so as not to disturb the sand. This was repeated several times and a large amount of brownish material was removed. A raked filter, after a normal run, cannot be washed in this manner. The loss of head on Bed No. 3 after this operation was only 2 in. and the effluent was satisfactory. As an experiment, the chlorination was stopped after a total of 110 days of pre-chlorination. Coliform bacteria were found in the effluent, the need of continuous chlorination being thus indicated.

When this bed was scraped, we found that the surface sand was lighter and easier to handle. Better separation of the dirt from the sand was obtained in the washing machine. The fine sand settled, whereas in normal operation the organic matter held it in suspension, so that much of it was lost in the wash water.

From these observations, we were confident that pre-chlorination had lengthened the run of a slow sand filter and had improved the appearance of the sand, without very much disturbance in the quality of the effluent.

Pre-chlorination of a Partially Clogged Filter with High Dosage Rates

A rapid method of controlling the chlorine dosage rate during pre-chlorination being necessary, a suggested procedure was to produce a chlorine residual in the filter effluent and to control operation by means of the ortho-tolidin test. The only filter available at the time was partially clogged but, because of the possible requirement of introducing pre-chlorination when the filters had been in operation for some time, we decided to experiment with it.

Bed No. 12 had been in service for 44 days and showed a loss of head of 28 in., indicating that it would have to be cleaned soon. Chlorine was introduced at the comparatively high rate of 15 ppm. The total residual chlorine at the point of application was 12 ppm and, at the far end of the filter, it was 5 ppm. The loss of head increased during the first few days and then started to fall off, until 8 days later it was 11 in. The increased loss of head was mainly the result of the death and settling out of numerous organisms, causing temporary clogging. This was apparently followed by chemical reactions that increased the porosity of the filter.

Sand samples taken at the beginning of the pre-chlorination showed a black fibrous layer on top with gray slimy sand underneath. As the chlorinated water penetrated the filter, the top layer became a brownish mud and the gray slime disappeared, leaving clean sand underneath.

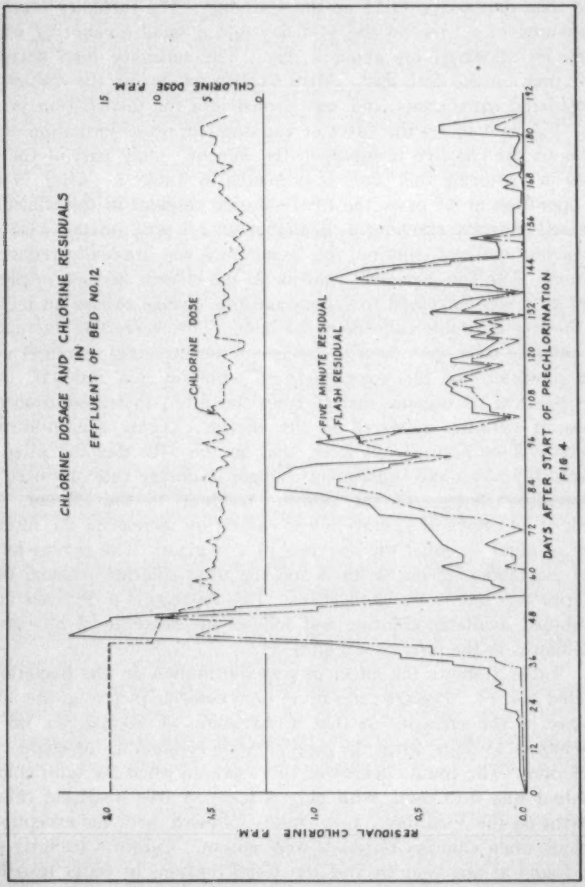
The effluent underwent a number of changes. At first, a slightly earthy or swampy odor was noticeable, which in a few days became objectionable. After 19 days of operation, a chlorinous odor was observed. The color increased from 5 to 30 during the first 34 days

and then diminished to 15 on the 48th day. The turbidity reached a maximum of 3 ppm on the 41st day and a small amount of whitish particles appeared for about a day. The turbidity then decreased to 1 ppm on the 46th day. After 54 days of curing, the effluent was considered satisfactory and was turned into the distribution system.

Figure 4 shows the effect of variations in pre-chlorination dosage rates on the chlorine residuals in the effluent. Only part of the data used in preparing this chart is presented in Table 2. After 34 days of operation at 15 ppm, the total chlorine residual in the effluent increased sharply, reaching a maximum of 2.4 ppm on the 43rd day. To avoid waste of chlorine, the dosage rate was gradually reduced to 5 ppm. The free available chlorine in the effluent became negligible. The dose was increased to 7 ppm and this change caused an increase in the free available chlorine to 0.1 ppm. The succeeding fluctuations in chlorine dose were largely due to our unsuccessful attempts under the conditions of this experiment to maintain this residual. With the removal of organic matter from the filter, increased concentrations of chlorine appeared in the effluent. There are indications, which will be pointed out later, that on the 84th day the filter was practically clean and subsequent changes in dosage rate produced corresponding changes of the chlorine residuals in the effluent. The peak at 141 days of operation was caused by increasing the filtration rate at night to equal the day rate of 2 m.g.a.d. The curves for the free available-chlorine residual and the total chlorine residual began to approach more closely together. This represents a decrease in the combined available chlorine and follows the decrease of nitrogenous substances in the filter (see Figure 5).

Table 2 shows the effect of pre-chlorination on the bacteriology of Bed No. 12. Dosage rates of 15 ppm chlorine increased the 20° C. counts in the effluent, so that a maximum of 70,000 per ml was reached in 15 days, when the total chlorine residual in the effluent was 0.05 ppm. The counts decreased to 10 per ml when the total chlorine residual was 0.18 ppm, with only a trace of free available chlorine present on the 33rd day. Low counts followed, with the exception of periods when chlorine residuals were absent. Coliform bacteria were not found at any time in 5 of the 10-ml portions of water tested.

This bed was used for 133 days, when it was scraped because of


CHLORINE DOSAGE AND CHLORINE RESIDUALS
IN
EFFLUENT OF BED NO.12.
CHLORINE DOSE P.P.M.
CHLORINE DOSE
FIVE MINUTE RESIDUAL
FLASH RESIDUAL
RESIDUAL CHLORINE P.P.M.
DAYS AFTER START OF PRECHLORINATION
FIG 4

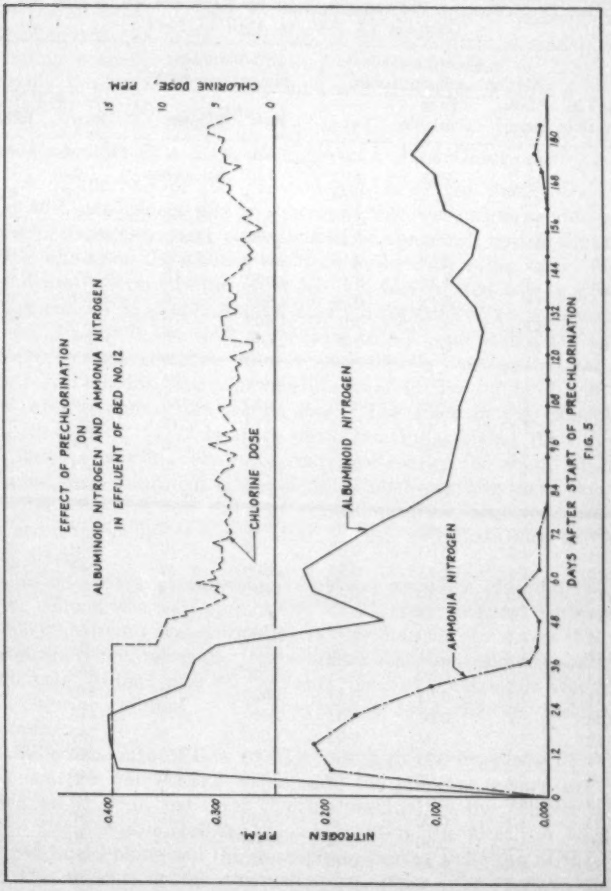

EFFECT OF PRECHLORINATION
ON
ALBUMINOID NITROGEN AND AMMONIA NITROGEN
IN EFFLUENT OF BED NO. 12
CHLORINE DOSE, P.P.M.
15
10
5
0
CHLORINE DOSE
ALBUMINOID NITROGEN
AMMONIA NITROGEN
NITROGEN P.P.M.
0.400
0.300
0.200
0.100
0.000
0
12
24
36
48
60
72
84
96
108
120
132
144
154
168
180
DAYS AFTER START OF PRECHLORINATION
FIG. 5

TABLE 2.—EFFECT OF PRE-CHLORINATION ON BACTERIOLOGY OF BED No. 12.
(October 15, 1947, to April 19, 1948)

Days in operation	Chlorine dose (ppm)	Chlorine residual in effluent (ppm)		Standard agar-plate count, 20°C.		M.P.N.	
		Free available	Total	Raw	Effluent	Raw	Effluent
1	15	0.00	0.00	1,000	7	< 2.0	<2.2
5	15	0.03	0.03	730	Numerous	9.8	<2.2
8	15	0.04	0.04	310	"	5.0	<2.2
15	15	tr	0.05	1,500	70,000	> 39	<2.2
23	15	tr	0.11	550	21,000	> 39	<2.2
29	15	0.03	0.15	9,000	7,000	> 39	<2.2
33	15	tr	0.18	3,600	10	> 39	<2.2
37	15	0.17	0.60	3,000	6	> 39	<2.2
41	11	0.70	1.0	1,600	0	15	<2.2
48	10	1.8	2.0	1,600	0	> 39	
54	4.3	tr	0.20	510	0	> 39	<2.2
55				490	800	15	<2.2
56	5.4	0.00	0.10	670	560	39	<2.2
57	5.2	tr	0.16	330	10	> 39	<2.2
62	5.0	0.10	0.40	410	0	15	<2.2
72	4.0	0.05	0.20	4,600	0	> 39	<2.2
82	4.5	0.70	1.0	1,200	0	> 39	<2.2
89	4.5	0.65	1.1	740	2	> 39	<2.2
96	6.1	0.80	1.0	720	0	> 39	<2.2
103	4.2	0.03	0.05	650	0	240	<2.2
110	3.5	0.25	0.35	3,600	0	240	<2.2
117	3.5	0.10	0.15	2,000	18	240	<2.2
124	2.5	tr	0.12	4,500	4	>240	<2.2
132	3.5	0.30	0.45	6,400	2	240	<2.2
138	3.5	tr	0.05	4,400	3	96	<2.2
145	3.0	0.60	0.75	2,200	1	240	<2.2
152	2.5	0.05	0.13	1,600	2	240	<2.2
156	1.2	0.00	0.05	6,000	600	>240	
159	3.0	tr	0.05	3,000	9	240	<2.2
161	2.3	0.00	0.05	12,000	1,400	240	
166	4.3	0.00	tr	1,800	840	240	<2.2
169	5.0	0.03	0.10	1,300	2	38	
173	3.1	0.00	tr	3,000	520	240	<2.2
180	4.5	0.00	tr	1,000	720	240	<2.2
182	6.0	0.25	0.40	950	3	> 39	
187	4.5	tr	0.05	1,700	7	> 39	<2.2

ppm—parts per million.

M.P.N.—Most Probable Number of coliform bacteria per 100 ml. A value of <2.0 indicates the absence of coliforms in a 1-ml portion and in all 5 of the 10-ml portions. A value of <2.2 represents the absence of coliforms in all 5 of the 10-ml portions.

tr—trace.

high loss of head. During its entire run, the flow of water through this filter was 393 m.g.a. Starting at the same time, a control filter operating normally filtered 118 m.g.a. The increase of 233% was partially due to the favorable influence of winter operation toward the end of Bed No. 12's run.

Pre-chlorination of a Scraped Filter with High Dosage Rates

A comparison of the pre-chlorination of the partially clogged filter with pre-chlorination of a scraped one was made, to determine if the time necessary for curing could be shortened and if the undesirable effects on the effluent could be avoided by scraping.

A newly scraped filter, Bed No. 10, was treated with a chlorine dosage rate of 15 ppm. A total chlorine residual of 15 ppm was observed at the influent well, decreasing to 4.5 ppm at the far end of the filter. The resulting effluent was slightly earthy in taste and odor. No coliform bacteria were observed in five of the 10-ml portions of water after the second day. The color in the effluent increased from 12 to 22 in three days, remaining higher than normal for about a month. The odor remained earthy for about 21 days, followed by a chlorinous odor. Small whitish particles were sloughed off after the first few days of pre-chlorination, and the turbidity reached a maximum of 4 ppm after 36 days and diminished to 1 ppm after 50 days.

After 7 days of operation, the total available chlorine residual in the effluent was 0.05 ppm. The 20° C. count reached a maximum of 600,000 per ml and gradually decreased to 12 per ml on the 23rd day of operation, when the free available chlorine residuals continued to increase, so that after 46 days they reached a maximum free available chlorine residual of 1.6 ppm and a total chlorine residual of 2.4 ppm.

When the rate of flow of water through the filter was decreased to a constant rate of 0.6 m.g.a. and the chlorine dosage rate was raised to 42 ppm, the total chlorine residual in the effluent was 9 ppm. Considerable frothing was observed in the filter. A haze developed in the plant and the atmosphere caused irritation of the eyes.

The chlorine dosage was reduced so that, after 72 days of pre-chlorination, the rate was 5 ppm, resulting in a satisfactory effluent.

The total flow through this filter was 420 m.g.a., representing a 100% increase in output of a comparable filter under normal operation.

The observations of Beds No. 10 and No. 12 indicate that pre-chlorination with high chlorine-dosage rates, as in super-chlorination, requires a curing period, which under our operating rates was about two months. During this period, the effluent may be unpalatable and the capacity of the filters would not be available. The slow curing of a scraped filter indicates that most of the organic matter was within the filter and that the deposit on a partially clogged filter was of lesser importance.

Significance of Ammonia Nitrogen and Albuminoid Nitrogen in the Effluent during Pre-chlorination

A relationship between the ammonia nitrogen and the chlorine residuals in the effluent is indicated in the results. Table 3 and

TABLE 3.—AMMONIA AND ALBUMINOID NITROGEN FROM PARTIALLY CLOGGED FILTER, NO. 12.

Date	Days in operation	Chlorine dosage (ppm)	Ammonia N (ppm)	Albuminoid N (ppm)
1947				
Oct. 23	7	15	0.198	0.396
Oct. 30	14	15	0.214	0.402
Nov. 6	22	15	0.176	0.404
Nov. 14	30	15	0.108	0.336
Nov. 21	37	15	0.018	0.324
Nov. 25	41	15	0.010	0.306
Dec. 2	48	10	0.008	0.146
Dec. 10	56	5.4	0.026	0.216
Dec. 16	62	5.0	0.008	0.220
Dec. 26	72	4.0	0.008	0.170
1948				
Jan. 2	79	4.0	0.002	0.128
Jan. 8	85	4.5	0.000	0.098
Jan. 16	93	3.5	0.000	0.088
Feb. 20	128	4.0	0.000	0.062
Apr. 9	176	5.0	0.012	0.124
Apr. 16	184	6.0	0.006	0.102

Fig. 5 show that the ammonia nitrogen in the effluent from Bed No. 12 reached a maximum of 0.214 ppm on the 14th day of operation and decreased thereafter to a low value of 0.018 ppm on the 37th day. The free available chlorine increased at the end of this period to maximum residuals.

TABLE 4.—AMMONIA AND ALBUMINOID NITROGEN FROM SCRAPED FILTER, NO. 10.

Date	Days in operation	Chlorine dosage (ppm)	Ammonia N (ppm)	Albuminoid N (ppm)
1948				
Apr. 20	1	15	0.000	0.060
Apr. 23	4	15	0.052	0.292
Apr. 30	11	15	0.094	0.338
May 7	18	15	0.180	0.510
May 14	25	15	0.140	0.510
May 21	32	15	0.040	0.340
May 28	39	15	0.014	0.290
June 4	46	15	0.028	0.278
June 11	53	42	0.024	0.344
June 18	60	19	0.014	0.244
June 25	67	17	0.024	0.248
July 9	81	8.0	0.036	0.194
July 16	88	6.5	0.002	0.158
July 23	95	6.3	0.012	0.142
July 30	102	4.5	0.016	0.140
Aug. 6	109	6.0	0.020	0.138

Table 4 shows that during the pre-chlorination of Bed No. 10 the ammonia nitrogen in the effluent reached a maximum of 0.180 ppm on the 18th day and then decreased to 0.014 ppm on the 39th day. The free available chlorine also increased at the end of this period to maximum residuals.

It is apparent that the ammonia was liberated in the filter by the action of the chlorine on material in the sand. When the ammonia nitrogen was at a maximum, practically no free available chlorine was observed in the effluent. When the ammonia nitrogen was at a minimum, high concentrations of free available chlorine appeared.

The concentration of albuminoid nitrogen in the effluent during

pre-chlorination is of significance in establishing the cleanliness of the filter.

Table 3 and Fig. 5 show that the albuminoid nitrogen in the effluent from Bed No. 12 reached a maximum of 0.404 ppm on the 22nd day of operation and decreased to 0.098 ppm on the 85th day. The albuminoid nitrogen in the raw water was 0.064. The low values for albuminoid nitrogen experienced thereafter indicate that the filter was fairly clean.

The albuminoid nitrogen in the effluent from Bed No. 10 is shown in Table 4. The albuminoid nitrogen reached a maximum of 0.510 ppm on the 18th day, decreasing until the 53rd day, when a minor peak of 0.344 ppm occurred because of the higher chlorine-dosage rate of 42 ppm. The albuminoid nitrogen then decreased, so that on the 88th day it was nearly constant at 0.158 ppm. The albuminoid nitrogen in the raw water was 0.160 ppm.

The albuminoid nitrogen, as a measure of the nitrogenous organic matter in the filter, shows that the partially clogged filter and the scraped filter were substantially alike in their behavior toward pre-chlorination, and that approximately the same time was necessary to clean them. We estimate that approximately 0.15 lb of chlorine were used to clean 1 cu ft of sand.

The Effect of Pre-chlorination on the Coliform Bacteria

The M.P.N. of coliform bacteria per 100 ml in the raw water and the effluent of the filters between November 13, 1947, and March 22, 1948, is presented in Table 5. The normally operated filters, Beds No. 1, 2, 4, 5, 8, 9, 10 and 11, were found to have coliform bacteria in the effluent. With continuing operation during the winter months of the year the M.P.N.'s decreased. Beds 8, 9 and 10 had just been scraped and the effluents had M.P.N. values of more than 16.1, which gradually decreased with continuing filtration. Toward the end of the run, when clogging was at a maximum, the effluents had fewer coliform bacteria. The effluents from Beds No. 1, 2, 4 and 11, whose filtering runs were completed during this period, had M.P.N. values of less than 2.2, 2.2, 5.5 and 2.2, respectively, at the end of the run. The M.P.N. of coliform bacteria became very small during the spring, as the water temperature rose and biological activity

TABLE 5.—M.P.N.* OF COLIFORM BACTERIA IN RAW WATER AND FILTER PLANT EFFLUENTS

Bed No.	1947 Nov. 13	Nov. 20	Dec. 22	Dec. 29	Jan. 5	Jan. 12	Jan. 19	Jan. 26	Feb. 2	1948 Feb. 9	Feb. 16	Feb. 24	Mar. 1	Mar. 8	Mar. 15	Mar. 22
1	16.1	>16.1	16.1	5.1	9.2	9.2	9.2	9.2	2.2	16.1	< 2.2	5.1	< 2.2	S	>16.1	>16.1
2	9.2	9.2	< 2.2	5.1	2.2	9.2	5.1	2.2	2.2	2.2	< 2.2	16.1	2.2	2.2	S16.1	>16.1
3[a]	>16.1	16.1	S16.1	5.1	>16.1	16.1	>16.1	9.2	5.1	9.2	9.2	9.2	2.2	2.2	5.1	2.2
4	5.1	>16.1	2.2	9.2	2.2	5.1	S	>16.1	16.1	9.2	16.1	>16.1	2.2	5.1	2.2	2.2
5																>16.1
6[b]	>16.1	>16.1	2.2	2.2	16.1	S	2.2	5.1	< 2.2	2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2
7[c]	2.2	16.1	< 2.2	5.1	16.1	>16.1	S>16.1	>16.1	>16.1	>16.1	>16.1	>16.1	>16.1	9.2	9.2	5.1
8	S>16.1	>16.1	9.2	16.1	5.1	9.2	>16.1	9.2	5.1	5.1	< 2.2	5.1	9.2	2.2	5.1	5.1
9	S>16.1	>16.1	2.2	2.2	< 2.2	2.2	2.2	9.2	9.2	2.2	9.2	>16.1	16.1	5.1	5.1	2.2
10	S>16.1	16.1	16.1	< 2.2	2.2	2.2	5.1	5.1	5.1	9.2	2.2	16.1	16.1	16.1	9.2	9.2
11	2.2	2.2	S 5.1	5.1	5.1	9.2	9.2	9.2	5.1	2.2	9.2	9.2	5.1	9.2	9.2	5.1
12[d]	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2
Raw water	>39	>39	>39	>39	>39	>39	>39	240	240	240	>240	240	>240	240	240	240
Post-chlorinated plant effluent	< 2.2	<2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2	< 2.2

*Most Probable Number per 100 ml of sample. A value of <2.2 indicates absence of coliform bacteria in all 5 of the 10-ml portions.

[a]Bed No. 3 pre-chlorinated July 10-October 31, 1947.

[b]Bed No. 6 pre-chlorinated January 14-April 10, 1948.

[c]Bed No. 7 pre-chlorinated November 25-December 18, 1947.

[d]Bed No. 12 pre-chlorinated October 15, 1947-April 19, 1948.

S—scraped.

increased in the filters. The post-chlorinated effluent from the plant was free of coliform bacteria in all five of the 10-ml portions of water at all times. The raw water had high M.P.N.'s throughout this period.

Bed No. 3 had been pre-chlorinated between July 10 and October 31, 1947. After scraping and return to normal operation, it behaved like a normal filter from the bacteriological standpoint.

Bed No. 6 was pre-chlorinated from January 14 to April 10, 1948. A few coliform bacteria were found during the first two weeks after scraping and none were found in five of the 10-ml portions of water thereafter.

The pre-chlorination of Bed No. 7 was too brief for definite conclusions.

Bed No. 12 was pre-chlorinated throughout this period and no coliform bacteria were found in its effluent at any time.

Summary

This investigation concerns the effect of the addition of chlorine to the raw water entering a slow sand filter and constitutes a radical departure from usual practice. The design of the Whitney Filter Plant allows experimentation without interfering with the normal operation of the plant.

Pre-chlorination increased filtering capacity, the increase depending on the condition of the raw water and the chlorine-dosage rates. The use of moderate dosage rates to give free-residual chlorination of the raw water increased the output of a filter 72% without disturbing the physical characteristics of the water. Cleaning of the sand in the filter by this method occurred at a very slow rate.

The use of higher dosage rates, as in super-chlorination, resulted in more rapid cleaning of the filter but necessitated waste of the unpalatable effluent for about two months. Approximately 0.15 lb of chlorine were used to clean one cubic foot of sand. Two filters cleaned by this method showed 100% and 233% increases in capacity, respectively.

During the cleaning period, the liberated ammonia was at a maximum when the effluent chlorine residual was negligible. The disappearance of the ammonia was followed by maximum chlorine residuals.

The albuminoid nitrogen in the filter effluents was shown to be of value in determining the cleanliness of the filter during high initial chlorine-dosage rates.

The coliform test was found to be the most satisfactory bacteriological procedure in controlling the sanitary quality of the effluent with pre-chlorination. The 20°C. standard agar-plate counts at first measured the rate at which the filter was being cleaned and later were of value in sanitary control, when the bacteria in the filter had been eliminated.

Gradual improvement in the effluents during pre-chlorination was observed, with the gradual removal of dirt from the sand and gravel *that had lain undisturbed since the plant was first placed in operation, 43 years ago.* In comparing the effluents from pre-chlorinated and normally operated filters, no claim can be made of improvement in taste, odor or appearance. Definite improvement in bacteriological quality was observed.

Acknowledgments

We wish to acknowledge the advice and encouragement of Mr. Edward E. Minor, Vice President of the New Haven Water Co., and the coöperation of the Wallace & Tiernan Co.

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DISCUSSION

ARTHUR C. KING (Taunton, Mass.). Thank you very much, Dr. Jacobson, for that very interesting paper. In these days of increased cost of labor I think the increase in the length of filter runs is quite important. Perhaps chlorine is cheaper than labor.

PETER C. KARALEKAS (Springfield, Mass.). Did you attempt to increase your filtration rates?

SAMUEL JACOBSON. Yes, we did; and we obtained about the same amount of water as when we operated at lower rates, using pre-

chlorination. That is, if we want to get our water in a hurry, we can do so and we shall get the same total amount.

MORTIMER M. GIBBONS (Rahway, N. J.). When you said pre-chlorination did not make the taste any better, did it make it any worse?

DR. JACOBSON. Well, at times, yes. We ascribe that to the low flows over the week-end. That is, the water remains stagnant under the filter and picks up some taste, which becomes concentrated in that way, so that it is noticeable during the lower rates of flow.

CHARLES R. COX (Albany, N. Y.). This paper is somewhat unique, because it discusses chlorination in connection with slow sand filtration. Almost all that the literature deals with is rapid sand filtration.

Some years ago—roughly about 1934—we were confronted with short filter runs at some of the slow sand filters in New York State. These were very old filters, one being the second oldest in the country. We were reluctant to suggest to these communities that chlorination be followed, because of the generally accepted theory that their efficiency depended upon the biological slimes in the sand, but, nevertheless, pre-chlorination was initiated at Hudson, N. Y., and later at six other plants in the state, with algae and organic control, with beneficial results. We did not have the breakpoint technique in those days. We started off with chlorine and gradually increased the amount of ammonia. In other words, we started off with very low doses over a long period of time and gradually increased the doses and the percentage of ammonia, so that we got more and more stable residual chlorine, until finally the results were secured. We now ignore the so-called biological slimes and use the benefits of pre-chlorination, getting much longer filter runs, comparable to the benefits of rapid sand filtration.

Ordinarily you can't discharge water to waste during this transition period. The method we have used—apparently with success, because we have not had so many complaints—is very gradually to increase the residual in the applied water and to use post-chlorination at the same time for any bacteriological control we might need in addition during that transition period, which of course is troublesome.

ALEXANDER J. MINKUS (Hartford, Conn.). We too have had

beneficial results by pre-chlorination of our slow sand filters. In our case we started with two beds, cleaned by the hydraulic method. We had one complete run. The free bed delivered 135 mil gal of water before it had to be scraped again, and the bed with the chlorine applied to the raw water delivered 292 mil gal before it reached the same loss of head. In our case we obtained a chlorine residual in the effluent in about 30 days' operation, and that residual at present is about 0.2. That is combined chlorine.

WARREN A. GENTNER (Hartford, Conn.). I think I might add to what Mr. Minkus has said by stating that our average rate of filtration on our slow sand bed is about 4 m.g.d.a., which is about double that of New Haven.

ONE HUNDRED YEARS OF PUBLIC WATER SUPPLY FOR BOSTON

BY JAMES J. MATERA*

[*Read November 18, 1948.*]

ON OCTOBER 25, 1948, the city of Boston commemorated the one hundredth anniversary of the introduction of its first public water supply.

One hundred years ago, on October 25, 1848, the Cochituate Water Supply System was officially dedicated to the service of the rapidly growing city of Boston and, for the first time in its history, the city had a supply of pure water adequate for its needs for many years, so far as could then be foreseen.

Previous to the year 1825, there was no public action on the part of the citizens of Boston to introduce, at the expense of the city, a supply of pure water, although it had been partially introduced by two private companies.

Dr. N. B. Shurtleff, in one of his articles on *Boston in the Olden Time*, gives a full account of the *Ancient Conduit* and also of the *Jamaica Pond Aqueduct Company*.

> Of the *Ancient Conduit* he says: "Among the most noted of the early attempts for procuring water for the daily use of the towns' people was the CONDUIT, a very singular contrivance, but one which answered a very good purpose in the limited space in which its benevolence was experienced. . . . In 1652, at the May Session of the General Court of the Colony, on petition of the inhabitants of Conduit Street in Boston, 'The Water Works Company' was incorporated for building the conduit, and provisions were made for the use of the water in case of fire.
>
> "From what has been stated, it would appear that the conduit was a large reservoir, about twelve feet square, made for holding water, conveyed to it by pipes leading from neighboring wells and springs, for the purpose of extinguishing fires and supplying the inhabitants dwelling near it with water for domestic purposes (see Fig. 1). Over the reservoir was a wooden building, in the olden time, used for storage purposes; but in more modern

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days, the old building was removed, and the conduit covered with plank, raised in the centre about two feet, and sloping to the sides, like a hipped roof. . . . This strange construction was situated in a square formed by the junction of Wing's lane (now Elm Street) and Union Street, in the neighborhood of the present North Street, and a short distance from Dock Square. The street leading from the conduit to the Draw Bridge placed over the Mill Creek, now the site of Blackstone Street, was one of the first highways laid out by the early settlers of the town, and was for a long time known as Conduit Street, because the proprietors of the conduit owned an estate on the north side of the street, about where the old building stands. . . .

"The old conduit never fulfilled the expectations of those who devised and built it; and its traces have so entirely disappeared, that not a single vestige of it can be found, and an occasional mention of the street that bore its name, and the old estate alluded to, is all that can be found concerning it in the ancient town books and in the conveyances of land in Suffolk Records. No digging in the street, for laying of drains or sewers, has, within the remembrance of persons now living, shown any of its remains, although it was well remembered in its last condition by the old persons who have recently passed away. . . .

"The exact position of the conduit is marked out on John Bonner's plan of the town, engraved in 1722, and has been pointed out by antiquaries as being near where the present North Street and Market Square join Union Street, just west of the 'Old Feather Store,' which was taken down between the 10th and 13th of July, 1860."

In another article, giving an account of the *Jamaica Pond Aqueduct,* he says: "On the 27th of February, 1795, Governor Samuel Adams approved an Act of the General Court whereby Luther Eames, Nathan Bond and William Page, and their associates, were vested with corporate powers for the management and direction of the business, as a Company, of bringing fresh water into the town of Boston by subterraneous pipes; and, by a subsequent Act passed on the 10th of June, 1796, this Corporation was empowered to assume the appellation of 'THE AQUEDUCT CORPORATION.'"

The Corporation was authorized "to bring from any part of the town of Roxbury into the town of Boston, and into any street in the same town, all such fresh water as they, the said Luther Eames, Nathan Bond and William Page, and their associates, or any, or either of them in their private and natural capacities 'then had or hereafter should' have a right to dispose of or to convey from the springs or sources thereof." The act gave power also to open the ground in any of the streets or highways in Roxbury and Boston as should be required for the sinking of the water pipes, but with very prudent provisions, which prevented the aqueduct from becoming a nuisance or impairing any right of the town of Roxbury or any of its inhabitants in and to the waters of Jamaica Pond.

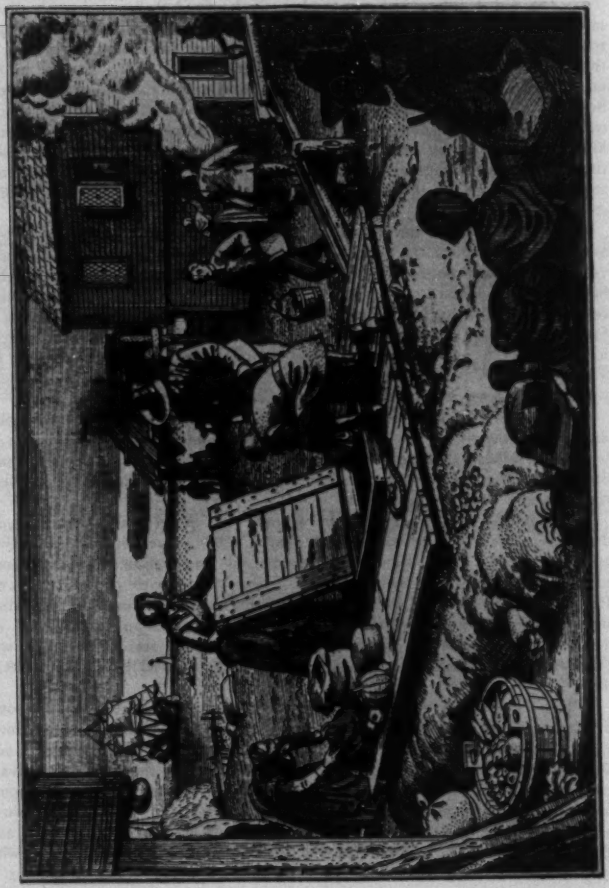


FIG. 1.—BOSTON'S FIRST RESERVOIR, BUILT NEAR DOCK SQUARE IN 1652.

"The Corporation could hold only thirty-three thousand dollars in real estate and the water works were to be divided into one hundred shares.

"The price of water was to be regulated by the General Court, the towns of Boston and Roxbury were to have the privilege of hydrants for extinguishing fires, and the first meeting was to be called by Hon. James Sullivan upon the proper application of the persons named in the act. On the 22d of June, 1803, an Additional Act was passed to facilitate the operations of the Corporation.

"The Capital of this Company, so far as can be ascertained, was about one hundred and thirty thousand dollars, or about thirteen hundred dollars to a share, which became much depreciated in value.

"No dividends were made during the first ten years after the commencement of the works, and subsequently the average of the dividends for thirty years amounted to a fraction less than four per cent a year. When the Aqueduct was in its greatest prosperity it supplied about fifteen hundred houses with water, chiefly at the South End, and in the neighborhood of Summer and Essex, and of Pleasant and Charles streets.

"The water was brought from Jamaica Pond in Roxbury through four main pipes of pitch-pine logs, two of four inches bore, and two of three inches, the lateral pipes having a bore of one and a half inch. The lineal extent of the water pipes in Boston was about fifteen miles, and they reached north as far as Franklin Street, and branched off easterly through Harrison Avenue into Congress Street nearly to State Street, and to Broad Street. They also branched off westerly through Pleasant and Charles streets, extending as far as the Massachusetts General Hospital, which was supplied with Jamaica Pond water. In 1840, this Company laid a 10-inch iron pipe from the Pond to Bowdoin Square."

In 1816 Cyrus Alger and others took some measures for the introduction of water into the town of Boston from Spot Pond in Stoneham, but upon further investigation they found it would be inexpedient. There was no public action on the part of the city government until 1825, on the subject of introducing pure water into the city. After 23 years of studies and debate, Boston obtained its first public water supply.

The Cochituate Water Act was passed by the State Legislature in 1846, authorizing the city of Boston to construct a public water supply, with an expenditure of about $3,000,000. The supply was to be taken from a natural lake, then known as Long Pond, situated in the towns of Framingham, Natick and Wayland. An aqueduct was to be constructed of brick masonry, with the necessary appurtenances to convey and distribute a pure and wholesome water supply to the inhabitants of Boston.

The introduction of the first water to the Frog Pond in Boston Common on October 25, 1848, was the subject of a monster celebration (see Fig. 2). To quote from the report published at that time:

WATER CELEBRATION

"The Water Commissioners having announced to the City Council, that the Aqueduct from Lake Cochituate to the reservoir and gate house in Brookline, would be completed, and that the main pipe for conducting the water thence to the City, would be laid on or before the 25th day of October, and would be in readiness for introducing the water of the lake into the City, on that day, a joint order of the two Boards was passed, directing an appropriate public celebration of the event, and appointing a joint Committee to arrange and superintend the celebration.

"The Committee resolved, as an appropriate form of celebration, to invite the citizens of the metropolis, with such of the public officers, the municipal authorities, the militia, various public bodies, and institutions of the Commonwealth, as might be disposed to accept the invitation, to unite in a procession, embracing a cavalcade and military escort, and to be present on the introduction of the the water at the Fountain, on the Common.

"The citizens cordially responded to the invitation, and they were joined by great numbers of their fellow-citizens from all parts of the State, who came to offer their sympathy on the joyful occasion. The usual avocations were suspended, in token of the common rejoicing in the accomplishment of a great and useful enterprise, and a vast concourse of citizens and strangers assembled to unite in the procession and to be witnesses of its progress and of the event to be commemorated.

"The day opened propitiously by beautiful weather after a heavy rain during the night, disappointing the gloomy anticipations of the preceding day. The discharge of artillery, at break of day, was the appointed signal that the celebration would proceed—as orders had been given for its postponement, in case the weather should prove inclement—and a salute of one hundred guns opened the ceremonies of the day, accompanied by the ringing of the bells of the City.

"At an early hour the streets were filled with people, attracted by the decorations, mottos, and devices, by which the principal avenues through which the procession was to pass were embellished. These were very numerous, and were well arranged, and in good taste, and some of them extremely beautiful.

"Several divisions were formed representing all groups of the young democracy. They paraded through the streets of Boston in a manner befitting the occasion. It took about two hours for all to pass a given point, ending the procession at the Common.

"Several addresses were made to the vast concourse of citizens assembled at the Common. At the end of his address, Josiah Quincy, Jr., Mayor

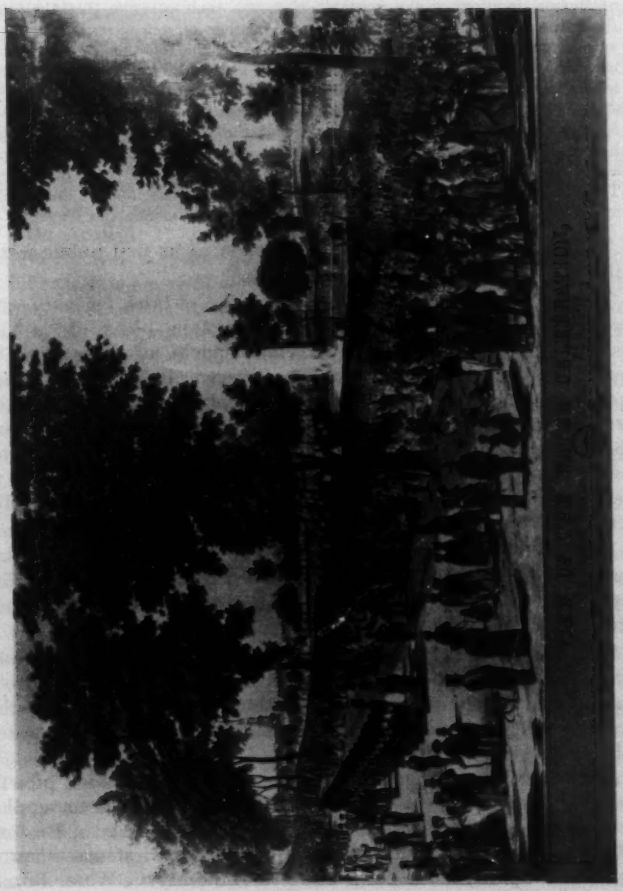


FIG. 2.—CELEBRATING THE INTRODUCTION OF COCHITUATE WATER INTO BOSTON, OCTOBER 25, 1848.

of Boston, inquired of the assembly if it was their pleasure that the water should now be introduced. An immense number of voices responded 'Aye', whereupon, on the signal of the Chief Engineer, the fountain gate was gradually opened and the water began to rise in a strong column, increasing rapidly in height, until it reached an elevation of about 80 feet. After a moment of silence, shouts rent the air, emphatically attesting the universal joy in witnessing this proof of actual attainment of the object of so long cherished hopes. The bells began to ring, cannon were fired, and rockets streamed across the sky. Men swung their hats and shouted, and some even wept. To the multitude around, the scene was one of intense interest and excitement, which it is impossible to describe, but which no one can forget."

Thus Boston celebrated the introduction of its first public water supply.

The Cochituate Aqueduct was constructed of brick masonry with cross-section dimensions 5 ft wide and 6 ft 4 in. high. It is 13.7 miles long and has a maximum carrying capacity of 20 mgd. The aqueduct terminates at the Brookline Reservoir and had a storage capacity of about 120 mil gal. Large iron pipes were laid from the reservoir into the city proper, which supplied a distribution reservoir on Beacon Hill, where now stand the capitol buildings of the Commonwealth. Additional distributing reservoirs were constructed in the east and south sections of the city. During 1869, Chestnut Hill Reservoir in Newton and Brighton was completed.

During 1849, the first complete year of operation of the works, the average daily consumption of water was 3.680 mil gal; in 1859, ten years later, it had increased to 13.175 mil gal, and in 1861 it reached 18.189 mil gal.

With the decision to develop Lake Cochituate for a water supply for the rapidly growing city of Boston, a policy of taking water from clean, unpolluted upland watersheds was established as being the most practicable method of providing a safe and acceptable water supply.

An increasing consumption of water and the continued growth of the city made it evident that provision had to be made to supplement the Cochituate supply. It was necessary for the city of Boston, in 1869, to make a contract with the city of Charlestown—then a separate municipality—to receive water from the Mystic Water Board to meet the demand. The city of Charlestown had established the

Mystic Water Board, for the purpose of developing a water supply from Mystic Lake, in 1861.

Although it had been apparent for some years to those intimately acquainted with the capacity of the existing works that the watershed of Lake Cochituate was insufficient to meet the wants of the city, it was not until 1871 that active steps were taken by the city government to seek a new source of supply. Studies and reports were then made which considered the Charles, Concord, Merrimack, Mystic, Shawsheen and Sudbury rivers as a source of supply.

On April 8, 1872, the Governor, W. W. Washburn, signed the Sudbury River Act for an additional supply from Sudbury River and Farm Pond in Framingham, thereby authorizing the Cochituate Water Board to divert the waters of the Sudbury River in Framingham. The Act provided for the construction of seven storage reservoirs on the river, together with a conduit 17 miles in length, now known as the Sudbury Aqueduct, through which water was to be conveyed to Chestnut Hill Reservoir, where provision was made to connect the Sudbury source with the existing works.

With the annexation of Charlestown, West Roxbury and Brighton in 1874, the Cochituate and Mystic Water Boards were united; and in 1876 the Boston Water Board was created, succeeding the Cochituate Water Board.

In 1875, construction of the Sudbury Aqueduct and Framingham Reservoirs Nos. 1, 2 and 3 was begun. The works were completed in 1880, although water was first delivered from the Sudbury River to Chestnut Hill in 1878. Ashland and Hopkinton Reservoirs were constructed on the South Branch of the Sudbury River.

The Sudbury Aqueduct is largely of the grade-tunnel type, of a horseshoe shape, having maximum dimensions of 9 ft in width and 7 ft 8 in. in height. With a minimum grade of a foot per mile, the aqueduct has a capacity of about 109 mgd.

In about 1890, it was realized that further additions would have to be made to supply the increasing needs of Boston. Also, the cities and towns around Boston began to feel the inadequacy of their water supplies, due to the encroachments that were rapidly being made on the watersheds of their sources, while, at the same time, the daily use of water was increasing beyond the limit of safe yield of these sources.

In 1893 and 1894, an intensive study was undertaken by the State Board of Health, relative to procuring a water supply for the city of Boston and its suburbs. This investigation was made under the direction of Frederic P. Stearns, Chief Engineer. It resulted in the report of 1895, which was the most thorough and comprehensive study ever made on the subject. Consideration was given to every large source of supply in the Commonwealth and to a proposal of taking water from Lake Winnepesaukee in the abutting state of New Hampshire.

The report recommended the creation of the Metropolitan Water District, with a proposal to go to the South Branch of the Nashua River for a water supply which would care for the needs of the Metropolitan District for many years to come. In his concluding remarks the Chief Engineer stated that the South Branch of the Nashua River was by far the best source from which to take an additional water supply, because it would furnish by gravity, with the least delay and the least cost, a very large quantity of pure water; it would be a step in the right direction further to augment this supply with a practically unlimited supply of pure water at a comparatively small cost. This recommendation was the original proposal in the plan for the future development of the Ware and Swift Rivers.

The principal features recommended for immediate construction were: a storage reservoir, by impounding the Nashua River; an aqueduct capable of conveying 360 mgd from the reservoir to the Sudbury watershed; another aqueduct, "Weston Aqueduct", capable of conveying 300 mgd from the Sudbury system to the Metropolitan District; pumping stations for Chestnut Hill and Spot Pond, and a system of supply pipe-lines to various cities and towns comprising the District.

The question of filtration was very thoroughly discussed at this time and it was concluded that, although an impure water could be rendered hygienically safe and of pleasing appearance—that is, clear and sparkling—reliance should be had in the first instance on a clean, unpolluted upland source, for which there was and still is a strong public preference.

In the discussion of quality, the question of quantity is most important. The average runoff from our southern New England

streams yields about 1 mgd per square mile of drainage area but, to make this yield available in the drier months of the year, large storage is necessary. In addition to providing a reserve in periods of drought, large storage also greatly improves the quality of the water. The organic matter, which contains the food for most bacteria and which is carried into the reservoir from the tributary streams, becomes ineffective. Sunlight and air have an opportunity to bleach and aerate the water when a large surface is exposed to the sun and wind.

The first meeting of the newly created Metropolitan Water Board was held on July 19, 1895. Frederic P. Stearns was appointed Chief Engineer, while Dexter Brackett was appointed engineer in charge of distribution and Thomas F. Richardson engineer of the aqueduct department. Joseph P. Davis, formerly City Engineer of the city of Boston, and Alphonse Fteley, Chief Engineer of the new water supply of the city of New York, were selected as consulting engineers.

Work started on the Wachusett Aqueduct in 1896. The first two miles of the aqueduct are a rock tunnel, followed by seven miles of masonry aqueduct, including a bridge over the Assabet River, and three miles of open channel. That portion consisting of the Assabet River crossing was replaced with the construction of an inverted siphon during this year, due to excessive leakage of the structure. The tunnel section is lined for about one-half its length with brickwork, 12 in. thick, and, where lined, is 12 ft 2 in. wide and 10 ft 10 in. high. The masonry section is 11 ft 6 in. wide and 10 ft 6 in. high, of horseshoe shape. The open channel is 20 ft wide on the bottom, with sloping sides.

The first contract awarded by the Board, on February 14, 1896, was for the construction of the Wachusett Aqueduct. Water was first turned from the Nashua River into the completed aqueduct on March 7, 1898, fifty years ago. Metropolitan Boston, therefore, celebrates a double anniversary during this year of 1948.

The Wachusett Dam, located in Clinton, is a gravity-type structure, 944 ft in length, including abutments at each end, crossing the valley of the river, with its top 20 ft above high-water level in the reservoir. Extending from the northwesterly end is a waste weir, 452 ft long, over which the flood waters are discharged into a channel

excavated in rock, 1,150 ft long and falling 107 ft to the river below. The main dam is composed of rubble masonry, quarried about a mile from the dam, with a facing, where exposed, of ashlar quarried in Chelmsford, Mass. The height at the point of deepest excavation is 207 ft, the maximum thickness is about 185 ft and, under the projecting cornice at the top, the thickness is 22.5 ft.

The North Dike, 10,000 ft long, and the South Dike, 2,800 ft long, were constructed of well-graded impervious materials, with cores of sheeting and concrete.

The Wachusett Reservoir has a watershed area of 107.69 sq mi. It is 8.4 miles long, with a maximum width of 2 miles, an area of 6.5 sq mi, a shore line of 37 miles, and a capacity of 65,000 mil gal at elevation 395. The maximum depth of water is 129 ft and the average depth is 48 ft.

The Wachusett Reservoir and Dam were substantially completed in 1906 and the reservoir was filled to a high-water mark for the first time on May 10, 1908. Generating equipment was installed in the lower gate chamber of the Wachusett Dam, and electric energy was produced for commercial use, for the first time, on August 10, 1911. This development of electric energy from a water supply was the first of its kind, as far as we are able to ascertain.

At the beginning of the Metropolitan Water Board's activities it purchased the various storage reservoirs and aqueducts of the Cochituate, Mystic and Sudbury systems from the city of Boston, as well as Spot Pond from the cities of Malden, Medford and Melrose.

Metropolitan Boston had a water supply second to none in 1910. At that time the general assumption was that the then existing facilities and supply of pure and sparkling water would be sufficient to serve many generations to come. However, in 1919, less than ten years later, the State Legislature provided legislation for an investigation of the water-supply needs of the Commonwealth relative to increasing the available supply. The joint boards of the State Department of Public Health and the Metropolitan District Commission were given the responsibility of making the investigations.

Relative to the supply for the Metropolitan District, the recommendations that were submitted in this report of January, 1922, were based on the original proposals and recommendations of the report

of 1895, which stated that the chief reliance of the future water supply of the Metropolitan District was placed upon the use of the Ware and Swift Rivers.

At this time the Metropolitan Water District included the city of Boston and 18 suburban cities and towns, as follows: Arlington, Belmont, Chelsea, Everett, Lexington, Malden, Medford, Melrose, Milton, Nahant, Newton, Quincy, Revere, Somerville, Stoneham, Swampscott, Watertown and Winthrop.

The total population which the District served in 1920 was 1,252,903, of which number 748,060 were included in the city of Boston. Examination of the census records at that time showed that the population had doubled in each period of 30 years, although for the 30-year period ending with 1920 the increase was about 89%. Therefore, an estimated population growth of 67% was assumed for the District for the next 30 years, ending with 1950. At the present time this figure will be closer to 50% .

Much controversy arose at the time regarding the report. In May, 1924, a report of the legislative joint standing Committee on Water Supply was submitted. Further divided opinion evolved from this report, relative to the subject of additional water supply for the Metopolitan District.

In August, 1924, an investigating commission was appointed by the Governor and it submitted its report in December of the same year. The investigating commission recognized the need of an additional water supply and recommended the immediate appropriation of $27,500,000, to be used by a newly created authority or commission in constructing the South Sudbury filters—population growth on the Sudbury watershed had created undesirable conditions in connection with sanitation—to develop the Ware River supply, to make necessary land and property takings in the Assabet and Ipswich River watersheds for future needs, and to make further studies and report to the next Legislature upon the subject of a pressure tunnel for the distribution of water in the District. This plan was not accepted.

The Metropolitan District Water Supply Commission was created by an Act of the Legislature in 1926 and was given additional power and duties in 1927. An appropriation of $65,000,000 was

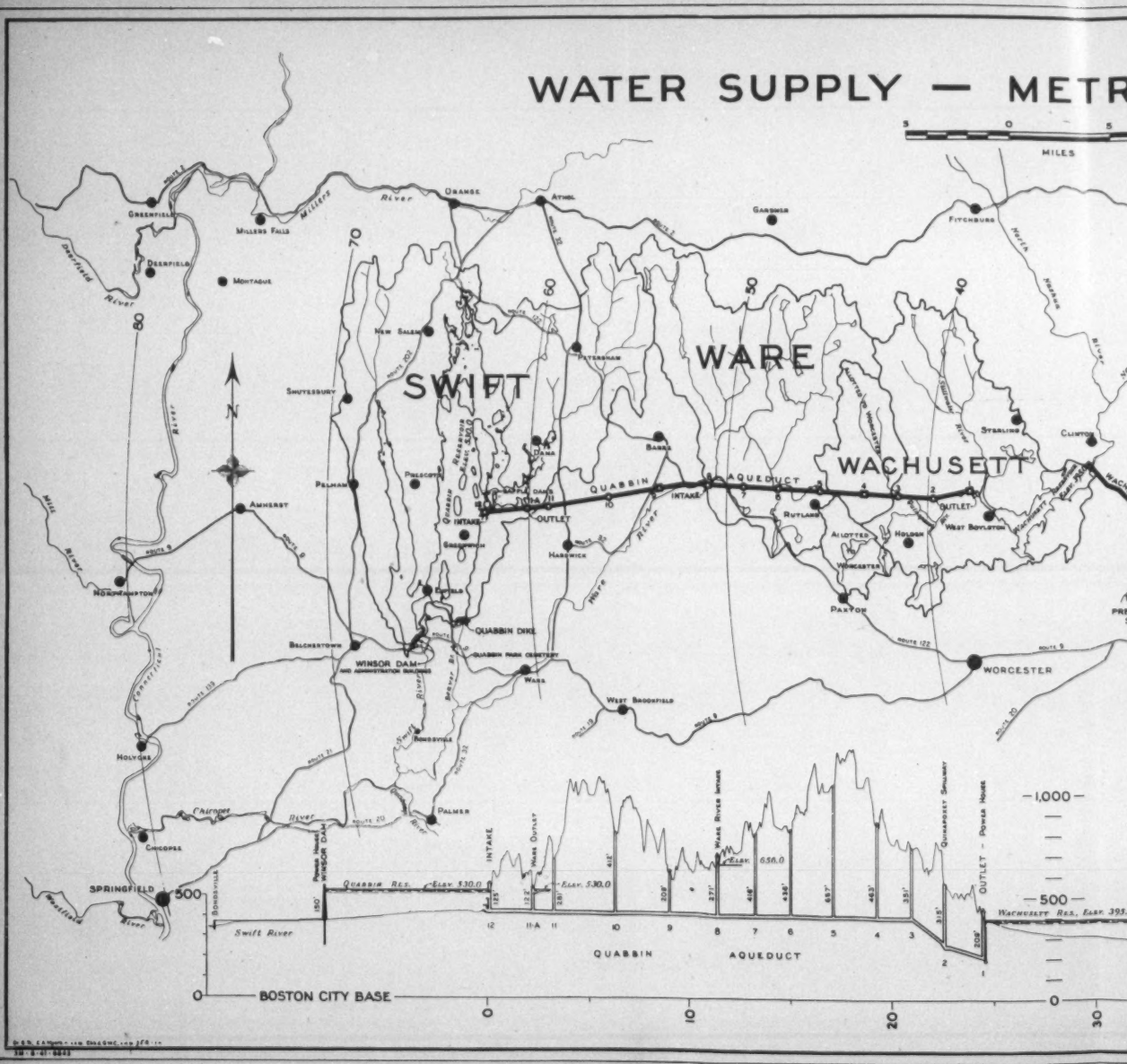
made for the development of the Ware and Swift River watersheds, as recommended in the report of the joint board in 1922. Frank E. Winsor was appointed Chief Engineer and X. H. Goodnough, Chief Engineer of the Department of Public Health, and J. Waldo Smith, formerly Chief Engineer of the New York Board of Water Supply, were selected as consulting engineers.

The Quabbin Aqueduct, 24.6 miles long, 11 ft wide, and 12 ft 9 in. high, of horseshoe shape, with a capacity of 975 mgd, was built from the westerly portion of the Wachusett Reservoir in the town of West Boylston to the new Quabbin Reservoir. A diversion dam was built on the Ware River, as well as other works necessary to provide for the diversion of water into the Quabbin Aqueduct. That portion of the Quabbin Aqueduct required to divert water from the Ware River is about 13 miles long and was completed in 1931, to permit the diversion of water into the Wachusett Reservoir for the first time on March 20, 1931. In addition to diversion in this direction, water may be diverted from the Ware River intake works westward, to be stored in the Quabbin Reservoir.

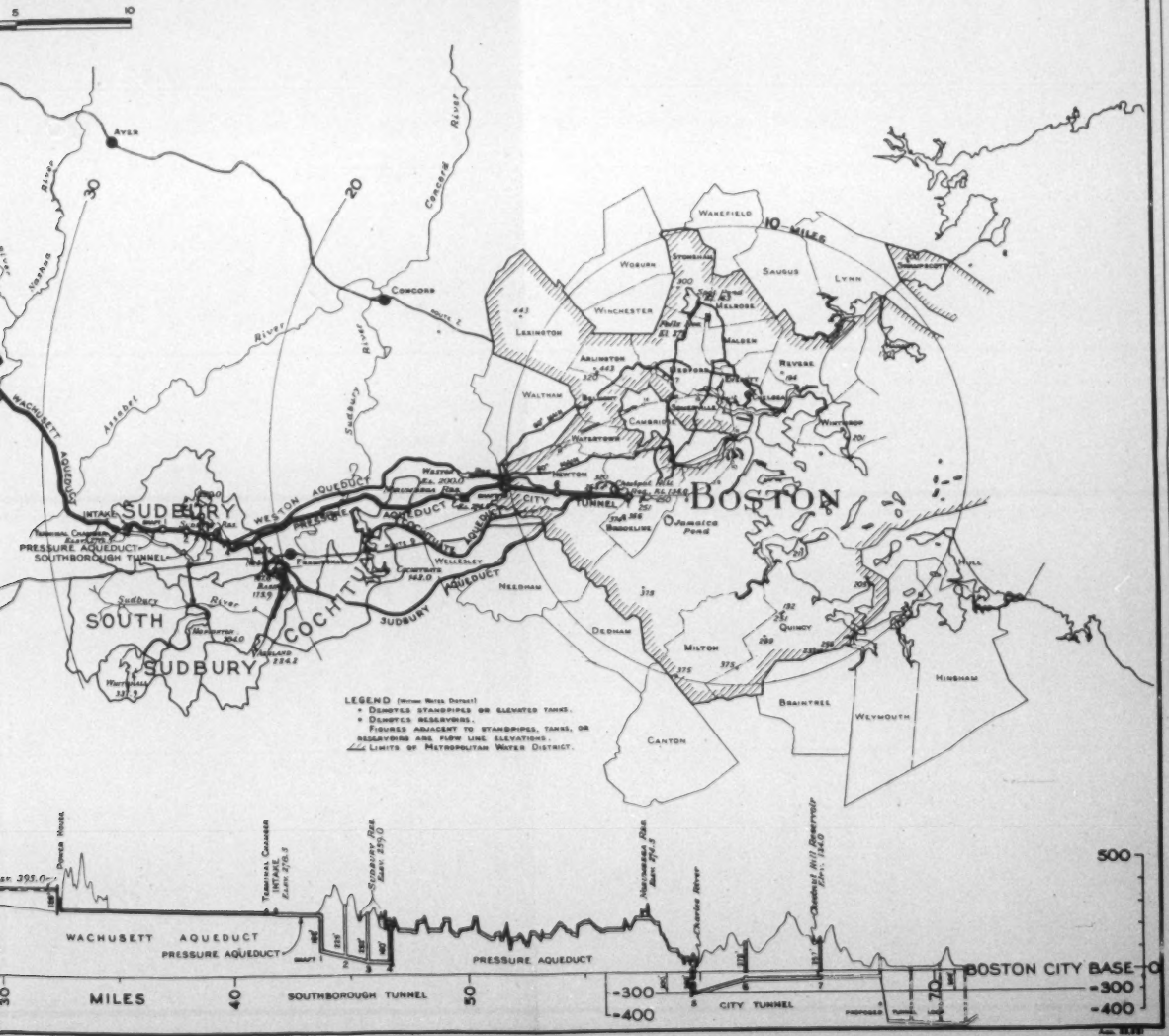
Quabbin Reservoir, about six times as large as the Wachusett Reservoir, was built in the valley of the Swift River, with a storage capacity of 412,000 mil gal at high-water elevation 530. It is 18 miles long, with a water surface of 38.6 sq mi and a shore line of 118 miles, and has a depth of 102 ft to the lower intake sill of the Quabbin Aqueduct.

The main dam, called Winsor Dam for the late Chief Engineer of the Water Supply Commission, is 2,640 ft long, 170 ft high above the bed of the river, and 295 feet above the sound-ledge foundation. It contains 4,000,000 cu yd of selected earth fill above the original surface. A spillway, 400 ft long, with its crest at an elevation of 530 ft above mean low tide, was cut through a rock gorge east of the dam and fixes the high-water elevation of the reservoir.

Quabbin Dike, located about 3 miles east of Winsor Dam, is 2,140 ft long, 135 ft high above the bed of the brook and 264 ft above the sound-ledge foundation, and contains 2,500,000 cu yd of earth fill above the original surface. Both dams were built with a concrete core-wall foundation to sound ledge and the fill embankments were built by the hydraulic-fill method, from borrow areas on land of the Commonwealth located below full-reservoir flow line.


WATER SUPPLY — METR
MILES
SWIFT
WARE
WACHUSETT
QUABBIN
AQUEDUCT
INTAKE
OUTLET
WINSOR DAM
QUABBIN DIKE
GREENFIELD
MILLERS FALLS
ORANGE
ATHOL
GARDNER
FITCHBURG
DEERFIELD
MONTAGUE
NEW SALEM
PETERSHAM
SHUTESBURY
PELHAM
AMHERST
NORTHAMPTON
BELCHERTOWN
PRESCOTT
GREENWICH
HARDWICK
BARRE
RUTLAND
HOLDEN
WEST BOYLSTON
STERLING
CLINTON
PARTON
WORCESTER
WARE
WEST BROOKFIELD
HOLYOKE
CHICOPEE
SPRINGFIELD
PALMER
THORNDIKE
Connecticut River
Deerfield River
Millers River
Swift River
Ware River
Chicopee River
Westfield River
Quaboag River
Nashua River
Assabet River
Wachusett Reservoir
Quinapoxet Reservoir
Boston City Base
Quabbin Res. Elev. 530.0
Wachusett Res. Elev. 395
Ware River Intake
Elev. 656.0
Quinapoxet Spillway
Outlet - Power House
Winsor Dam Power House
Quabbin Park Cemetery
Route 2
Route 9
Route 122
Route 20
Route 32
Route 202
Route 101
Route 32A
Route 122A
Route 139
Route 31
Route 12
Route 140
Route 62
Route 68
Route 70
Route 56
Route 135
Route 122
Route 30
Route 19
Route 8
Route 63
Route 67
Route 49
Route 12
Route 122
Route 9

TROPOLITAN BOSTON



The impounding of water in Quabbin Reservoir was begun on August 7, 1939, and on May 30, 1946, water reached the crest of the masonry spillway at elevation 528. On June 22, 1946, a celebration was held, commemorating the filling of Quabbin Reservoir.

With the completion of the Ware-Swift River development for the Metropolitan Water Supply, the Commonwealth of Massachusetts had constructed one of the greatest engineering projects undertaken in this part of the country (see Fig. 3).

In 1938 an extensive report was made by the Department of Public Health, working in conjunction with the Metropolitan District Water Supply Commission, relative to improvements in distribution and adequate prevention of pollution of sources of the water supply for the Metropolitan Water District. It dealt mainly with the pollution problems in the Cochituate, Sudbury and Wachusett watersheds, created by increased population and industrial activities on these watersheds.

Several recommendations were made for the elimination of sources of pollution, which included the discontinuance of the regular use of water derived from the Cochituate and Sudbury watersheds, although use of the Cochituate supply for consumption in the District ended during October, 1931. Another recommendation of the report was the immediate construction of a pressure aqueduct, bypassing the Sudbury system and connecting with the distribution system, with a long-range plan for a tunnel loop, which would reinforce existing supply mains, thereby eliminating considerable pumping.

On the basis of these recommendations the construction of the pressure aqueduct, now known as the Hultman Aqueduct, was undertaken by the Metropolitan District Water Supply Commission. The Hultman Aqueduct was first placed in service during October, 1940, making it possible to bring water from the open channel of the Wachusett Aqueduct directly into the distribution supply mains for District consumption—a distance of approximately 18 miles.

Now under construction is a portion of the proposed city tunnel, which starts at the terminus of the Hultman Aqueduct in Weston. This section will be approximately 5 miles long, ending at Chestnut Hill Reservoir in Brighton. When completed, existing pumping at Chestnut Hill for the Southern high-service distribution will be eliminated.

During 1947 the Legislature gave authority to the Metropolitan District Commission to transfer to the Department of Conservation certain reservoirs in the Cochituate and Sudbury watersheds that were no longer used for water-supply purposes. Included was Lake Cochituate, from which no supply had been taken since October, 1931, as already mentioned.

On June 30, 1947, the Metropolitan District Water Supply Commission was abolished and from it was created the Construction Division, with Karl R. Kennison continuing as Chief Engineer and Stanley M. Dore as Deputy Chief Engineer. All completed work was turned over to the Water Division of the Metropolitan District Commission for operation and maintenance.

It is very interesting to note that the present vast system, capable of delivering about 300 mgd of water, has developed within a century from the Cochituate supply, which was originally planned to furnish less than 5% of the present supply.

The Metropolitan District Commission is composed of William T. Morrissey, Chairman, William F. Rogers, Joseph McKenney, Max Ulin and John J. Grigalus. Approximately 471 civil-service employees carry out the duties required for the operation, maintenance and improvement of the water supply which is the responsibility of the Water Division, of which Harold J. Toole is Director and Chief Engineer and Charles O. Clark is Deputy Chief Engineer.

Numerous articles and papers describing details of the water supply of the Boston Metropolitan District have been presented in the past to this Association, many of which have been published in the JOURNAL.

ALGAE CONTROL AT DANBURY, CONNECTICUT

BY ELLIS A. TARLTON*

[*Read March 17, 1949.*]

Two main systems of storage reservoirs, known as the West Lake and the Margerie Reservoir systems, supply Danbury through two similar 5-mgd rapid sand filtration plants. They are about equal in size, with a total storage capacity of about 3 billion gallons, and each main reservoir is supplied from three auxiliary reservoirs by gravity flow and by pumping, each serving about half of the city. The chemical and physical attributes, as well as the general treatment problems, of the two systems are quite different, although both are, in general, soft, colored surface waters.

The industrial demands of the city determine to a large extent our method of operation. Since our pure-water storage is only about 20% of the normal daily consumption and the demand during the day exceeds our treatment rate by about a third, we must operate so that, when our load comes on at 7:00 A.M., all filters are clean and all clear wells full. As plant design makes necessary a complete shutdown of at least an hour's duration to backwash filters, filter runs must be at least 10 hours, during which time the filtration rate must not fall below a minimum of 1.6 gal per sq ft of filter area per minute.

The felting and dyeing processes of our major industries demand a water free of color, turbidity, iron and aluminum, and with low pH and hardness, both held to a constant value. A weak floc, induced by large algae concentrations, will pass appreciable amounts of these interfering substances through the filters. Daily "blooming" of algae in our shallow reservoirs causes the raw-water pH to vary from 7.0 to 10.0 in a few hours. To insure safe operation and acceptable water quality, we have set up minimum standards for raw, treated and settled water at both our filtration plants, and control is merely a a question of operating so as to remain within these limits.

*City Chemist, 34 Pleasant St., Danbury, Conn.

Copper sulfate, for algae control, is applied by hanging bags of the chemical over the sides of a motor-boat and following a systematic course to give adequate distribution. This is obtained by following the reservoir shore-line as closely as possible for the first circuit and then making each succeeding circuit 25 to 50 ft within the previous one. We have been able to apply a maximum dose, in one day, of 4,000 lb by this method in warm weather, and about 2,500 lb in cold weather. We figure dosages from the total volume of water actually in the reservoir at the time of treatment, as computed from the original storage curves. Dosages range from 2 to 5 lb per mil gal. With each treatment of a given organism under similar conditions we attempt a lower unit dose.

We have never experienced a treatment that was ineffective, although we have had to wait as long as 10 to 14 days for its full effect to be felt at West Lake. As our control keeps us well ahead of trouble, such a delay is not a cause for worry. Usually conditions clear up in one to two days at the Margerie Reservoir and four to six days at West Lake.

Throughout the year we run weekly microscopic counts on the two main reservoirs, with counts daily or every other day during the critical months of June to October. Only five organisms are counted at the West Lake Reservoir, while at the Margerie Reservoir all organisms are counted and a careful watch is kept on the increase of any organism existing in appreciable amounts. Examinations of the auxiliary reservoirs are made occasionally, particularly before it is proposed to discharge one into the main reservoir. Such records, plotted on semi-log paper and interpreted in the light of past experience with the individual reservoir, together with other factors of present conditions and future plans, take the guesswork out of operations. By systematic biological examinations we are usually ahead of any taste and odor trouble. Waiting until a slight taste is detected in the filtered water makes it certain we shall experience one or two weeks of real taste and odor. This, of course, applies particularly to the West Lake supply.

The Margerie system consists of the main storage, Margerie Reservoir, which is on the site of an old swamp and contains large shallow areas, with a depth at the deepest point—at the intake—of

only 20 ft, and into which the large East Lake Reservoir feeds by gravity. East Lake has no shallow or swampy areas and the algae record is good. Normally the entire output of this reservoir is used first. The so-called King Street Diversion poses another problem, as it has no storage but a large, swampy catchment area, capable of seeding the main reservoir. Even in very dry weather this has a yield of 1 to 2 mgd. Unless we have an emergency, it must be allowed to run to waste during the summer months. It is an ideal location for continuous copper-sulfate application, and we hope that we shall have the manpower to treat it in that way this coming season and maintain its flow into the Margerie Reservoir the year round. The third auxiliary supply is the small Padanaram Reservoir, which has little storage, but a large catchment area and a high yield during wet weather. Since the cost of pumping is involved, this is used only when all supplies are low, after a prolonged dry spell.

The Margerie Filtration Plant, that treats all the water delivered by this system, uses alum coagulation and has an effective battery of spray aerators, rapid mix, flocculators, well-baffled twin settling basins with a minimum detention period of 7.5 hr, and six filter units, all equipped with surface-wash equipment and two of them capped with Anthrafilt. Lime and Calgon are used for corrosion control at a delivered-water pH of 8.0. Post-chlorination and ammoniation are used for sterilization, while provisions are made for pre-chlorination. A twin, mobile feeder unit has been a help in experimental feeding of various types of weighting agents, acid, ammonia, and silica at various points.

The water has about twice the hardness and mineralization of the West Lake supply, hign color and turbidity, and tremendous algae growths. It is not too corrosive, and the main problems in operation are providing a water that can be readily pretreated, so that no undue load is thrown on the filters, keeping the filters clean, and keeping odors down in the plant, so that effective chlorination without iodoform tastes can be maintained.

Since we are not bothered as much by taste and odor problems in the raw water at the Margerie Plant, as we do not pre-chlorinate and many of the tastes and odors readily filter out, and we have more equipment to attack the algae problem in the plant itself, we can view

algae control from a different angle here. That is, much of the time we can let growths continue that would swamp the facilities of the other plant. At the Margerie Plant we can take full advantage of modern equipment to achieve an economically balanced algae-control program. The West Lake supply we must continue to handle as if we were delivering directly to the city, without the benefit of a treatment plant. It is not economically justifiable to keep all algae growths in subjugation, when it is possible to handle them easily and cheaply at the treatment plant and deliver a clear tasteless water.

This is particularly desirable, since the main Margerie Reservoir is difficult to treat with copper by our primitive methods, due to the shallow areas, the stone walls running at random, the type of labor employed, and the fact that a new growth is well started a few days after one is successfully killed. Therefore, our plan of attack at Margerie has been to study algae counts carefully and to note whether one or more types predominate, what percentage of the total count they represent, and the rate of change from day to day of those types. As they increase, small daily adjustments in chemical feed, equipment, and operational methods keep pace with them, to maintain our predetermined optimum conditions, such as a maximum turbidity of 2.0 on the filters, a maximum pH of 6.7 in the treated water, minimum filter runs of 23 hr, minimum rates of flow of 70,000 gal per foot head loss per filter, and a minimum floc size in the last flocculator of 2.0 mm.

When we find the increase in the cost of chemicals and power to maintain the above conditions can justify an expenditure for copper sulfate, or the floc is ceasing to break in clear particles in the flocculators and is appearing in a greenish mist, or the pH cycle of the raw water, which has been ranging from 7.0 to 9.0 or 10.0 and recovering daily, holds its high peak through the second day, then we plan a copper-sulfate treatment immediately. Such control will average less than one treatment a month from July through October, for by picking our spots in this manner we can deliver our killing blow at the last possible moment.

Normally we obtain a large, fragile floc particle quickly, without agitation. Although the designer included flocculator equipment, its inclusion is not economically justifiable, as it is not needed to

build up a large floc particle, it is not used most of the time, and much of the time it actually reduces the particle size. However, we use the flocculators with certain growths—particularly *Coelosphaerium, Anabaena* and other free floaters—for we find that, while we get a smaller, poorer floc, we get settled water of better quality—that is, lower turbidity and color applied to the filters—which is, after all, what is wanted.

Jar tests are of little use, since the floc forms so readily that it is impossible to get results comparable to plant operation. It is only in times of trouble and winter operation, and in experimental work with coagulant aids using minimum alum dosages, that it is possible to draw worthwhile conclusions.

One of the most interesting features of the Margerie Reservoir is the daily pH cycle resulting from the daily "bloom" of algae growths in hot sunny weather, which uses up the free and half-bound carbon dioxide in the process of photosynthesis. Normally a pH of about 7.0 will prevail for half a day, when a gradual rise to below 8.0 will suddenly zoom to 9.0 or 10.0, as the growth reaches its highest point of logarithmic increase, holds this for two to three hours and drops off slowly, as the supply of carbon dioxide, released by the forces of decomposition, exceeds the demand and again depresses the pH.

In the early days of plant operation, nothing was known of the activity of these forces and the operators were continually puzzled by the sudden inexplicable shift to a settled water of poor quality. During the high peak of the pH cycle there was insufficient alum to lower the pH to the point of floc formation. If the algae growth is not checked, the daily variations gradually cease and a continuously high pH is found. With certain types of algae it has been found possible to continue operating without reservoir treatment, as long as costs are not excessive and there is no danger of losing control of floc formation. Subsequent to successful killing of a predominant organism we occasionally experience large growths of several types, reaching proportions of 5,000 to 10,000 v.s.u. Such large coincidental growths have usually been easy to pretreat, forming a fairly strong floc particle with moderate alum dosages.

We have found that, while an increase of the alum coagulant

is required to handle an increase in algae concentration, the large dosages necessary at times of high pH are in large part merely for pH depression to the optimum range. Such lowering of the pH of the raw or settled water may also be achieved by bypassing the aerators and retaining in the treated water the carbon dioxide released by the alum application, or by treating directly with sulfuric acid. Fortunately, odor control by aeration is unimportant and, therefore, the aerators are usually bypassed at the first sign of high pH, until about one day after treatment. If left off, there would be a sudden increase in treated-water acidity as soon as the first effects of the copper sulfate were felt, which would necessitate an immediate increase in post-lime treatment, and this sudden increase in hardness would interfere with the time schedules in the felting process in the hat factories. We are about to install continuous pH-indicating equipment at four points in the plant, to give continuous pH values of the raw-water supply, water in the flocculators, applied water, and effluent, as well as equipment for accurately feeding sulfuric acid, to counteract raw-water changes immediately and maintain an optimum pH in the treated water.

Although we understand trouble is met with in copper-sulfate treatment at high pH values, we have not yet experienced any. We continually get good killing in one to two days and complete disappearance of the organism in four to five days after having treated with 2 to 3 lb per mil gal at a pH of 8.0 to 10.0.

After our first year's experience with these high pH values, we made plans for using during the next season a ferric-sulfate coagulant, which operates best at such a high pH range. We had one month's experience during a heavy run of *Anabaena* with a raw-water pH ranging from 8.0 to 9.0. Results were very poor. It was necessary to feed lime with the ferric sulfate, but it was impossible to keep the pH at a consistent level through the plant. Even though the pH after introduction of the iron coagulant was about 8.5, it would drop to 7.5 on the filters and below 7.0 in the filtered water. The effluent averaged 1 to 2 ppm of turbidity, a color of 20 and an iron content of 0.5 ppm, values as high as 1.0 ppm being obtained at times. Prechlorination of 1.0 to 2.0 ppm did not help the situation. A return to a high alum dosage immediately cleared things up.

Continuous copper-sulfate application to the plant influent looked promising, due to the quick reaction in floc formation always experienced within an hour or two of treatment of the main reservoir, but has failed of fulfilment. Dosages as high as 2.0 ppm of copper sulfate had no effect. It is felt that the copper sulfate should be introduced at a point considerably in advance of the alum, so that the killing effect of the copper sulfate will be well underway before floc formation is begun. Our next attempt in this line will be at the intake, where we shall have the benefit of a contact period in about a quarter of a mile of 30-inch pipe.

Super-chlorination of the raw water was attempted two years ago. We had a strong growth of *Coelosphaerium* to work on at a pH of about 8.0, and had no difficulty in effectively killing the organism and changing the green color of the raw water to a drab grey, with a dosage of about 3 ppm. This gave a residual of about 0.5 ppm of free chlorine after contact in the settling basin. There was some improvement in floc formation and in the quality of applied and filtered water, but it was felt that a longer contact time with chlorine before alum application would be helpful here, too. Also, our filters were not in the best of condition at that time, as it was the end of a summer with our usual, ineffective, filter back-wash conditions, and the principal effect of the chlorine was to unload the filters, with consequent bad tastes and odors in the water.

Since then, in anticipation of further experiments and to remove a dangerous situation, we have installed surface-wash equipment on all filters. We now have really clean filter sand at all times, without mud banks or signs of mud balls. Operation in the late summer and early fall was not safe with back-wash equipment that gave 30% expansion and a four-minute wash to a filter. An accumulated load of alum sludge and mud remained on the filter sand at all times. It increased during the summer and fall, and was never entirely removed during the better back-wash conditions existing in the winter. Part of our program of algae control was providing equipment to maintain our filters at maximum efficiency at all times, to safeguard the quality of the water, so that unexpected troubles would not find us with our equipment only partly effective.

Another part of our algae-control program, that involved in-

creasing the filters' efficiency, consisted of capping the filters with Anthrafilt—a coarse filter medium that permits deep floc penetration of the filter instead of surface matting. So far, an experimental installation indicates that about double the amount of water may be filtered per foot of head loss, as compared with the other sand filters. This applies to both summer and winter conditions. We are only awaiting the end of our year's trial, to see how the Anthrafilt layer has stood up, before capping all filters.

Algae control at the Margerie Plant is not a factor of winter and spring operation. The summer months of July, August and September, as well as the month of October, pose the main algae problems. Not yet have we had to treat for algae in the spring. Our only fall problem is insuring good winter conditions and it is possible, from a study of the algae counts during November, to predict good winter conditions and thus avoid a last copper-sulfate treatment.

Any discussion of algae control always brings up the question of their acquiring resistance to copper-sulfate treatment. The fact that we are today getting as good results with smaller dosages than used previously would seem to answer that in the negative. We are of the opinion that algae treatment of any kind, resulting in artificial conditions due to killing off growths of algae, will upset natural biological equilibrium and will make subsequent treatments necessary. This is only true for deep reservoirs and where there is no seeding. Our East Lake Reservoir is a good example of natural equilibrium. Untreated and unseeded, it seldom contains more than a few diatoms.

Subsequent to most treatments we run laboratory determinations on the amount of copper in the water. Traces will remain in the raw water for about five days. About one-half this amount reaches the filters. We have never yet discovered a trace of copper in the filter effluent. We have found that such information is valuable in contacts with industries, the medical profession, and occasionally the public.

The West Lake system is made up of the main West Lake Reservoir, which has no swampy tributaries and few shallow areas, and into which the Boggs Pond Reservoir flows by gravity. Boggs Pond is also fairly deep and is located in a heavily wooded area. As it is on the West Lake watershed, it is normally kept full and permitted to overflow. The two Kohanzie Reservoirs are shallow and water

must be pumped from them to the West Lake Reservoir. As they tend to seed West Lake with algae, they must either be used before a copper-sulfate treatment is scheduled or be treated before use. The total storage of this system is the same as the Margerie System—about 1,500 mil gal.

The water is soft, of low mineralization, color and turbidity, and was used for years without any treatment other than chlorination. It was always famous, however, for its bad tastes and odors. Since the watershed is not heavily inhabited and all private sewerage systems are inspected annually, and are under the control of the State Board of Health and the local Water Department, there are no dangerous health hazards. The main problems are taste and odor control, corrosion control, and maintaining economical filter runs.

The West Lake Filtration Plant consists of two aero-mix units, giving some taste, odor and carbon-dioxide removal following alum coagulation, twin settling basins with 4.5 hours' minimum detention time, six filter units operating at 2.0 gal per sq ft per min, pre-chlorination and post-chlorination, with Calgon used for corrosion control at a delivered-water pH of about 6.0. The pre-chlorination plant, which safeguards about 30 families still using raw water directly, can be visited only once a day and so makes taste and odor control difficult, as the safe chlorine dosage for maximum flows gives super-chlorination dosages on the night flows, tending to unload the filter sand. So far, it has not been advisable to practice super-chlorination and de-chlorination on a city-wide scale continuously, although considerable experimentation has been done. Other plant facilities not at present in use are lime and activated-carbon feed, and ammoniation.

From the time the ice goes off in March a careful watch is necessary at West Lake, to keep ahead of taste troubles. With the first increase in water temperature we are apt to find *Dinobryon* present. *Tabellaria, Asterionella* and *Synedra* we carry through the winter without much change, assuming treatment was properly handled in the late fall. Usually one of the above will begin a slow increase, while the other growths remain static. The above diatoms will cause no taste trouble below 500 organisms per ml or filter trouble below 1,000. If they are in the ascendancy, we usually find treatment necessary

along in May with an appreciable increase in temperature. *Dinobryon* causes taste troubles at levels above 300 organisms per ml and will require treatment sometimes as early as the first week of April.

By using activated carbon we can delay reservoir treatment for several weeks but, if carbon is applied at its most effective point—directly on the filters—we get a decrease in filter runs by one-half within a week. Carbon applied at the mid-point of the settling basins is not so effective and plugs the filters almost as quickly, while even very large dosages in the raw water are ineffective. Principally due to pre-chlorination, taste removal in the plant has not been successful.

During our summer period of July, August and September we experience growths of *Protozoa, Cyanophyceae* and *Chlorophyceae,* but the above-mentioned four organisms continue as our problems. Fortunately, they do not increase rapidly and a maximum of two treatments during these months has kept conditions good. Our interest during this period is solely in taste control. Since the temperature of the water is at a maximum and our chlorine dosages are also at a maximum, this is our most difficult time to deliver a palatable water. It is mandatory that we now observe our maximum limits of algae concentrations for treatment.

The fall period, from October to freeze-up in December, usually gives us our most active growths, but taste is not pronounced and we therefore treat only to give efficient filter runs. In late November and early December our major effort is to prepare for winter operation, so that we shall not be caught by algae growing under the ice, that will shorten filter runs and give tastes. If our so-called final treatment of about 2.5 lb per mil gal is applied early in November, we are almost certain to have an upsurge of another algae growth before freeze-up. By holding off until late November before treatment, there will usually be no more growths, although we watch carefully until ice forms for such an increase in another type. In the past eight years this has happened twice. Once, after *Asterionella* treatment we neglected a new growth of *Tabellaria* as insignificant and suffered from a 50% decrease in filter runs the whole winter season. On the other occasion, we killed off a similarly increasing *Tabellaria* growth and had practically sterile conditions, in so far as algae were concerned, the entire winter.

REPORT OF COMMITTEE FOR SURVEY OF GROUND-WATER SUPPLIES IN NEW ENGLAND

This committee, consisting of a representative of each of the New England states and the U. S. Geological Survey, has attempted to gather available information concerning public ground-water supplies in the several states. Most of the aspects of the subject of "Ground-Water Supplies" suggested in the 1940 report of the Committee on Committees have been covered to some extent in the prepared tabulations.

A previous report, including similar tabulations, was submitted in 1941. These data have now been brought up to date.

The tabulations have been made as similar as possible. All of the systems listed are classed as "public" and supply water to 25 or more customers throughout the year.

There are many analyses that might be made of the tabulations and anyone with an interest in some particular phase of ground-water development will probably desire to make his own analysis of the information listed. It is believed, however, that some general summary and comments may be of interest.

There are over 300 water systems listed which are supplied wholly or in part from ground-water sources. These systems supply about 1,677,000 persons.

A simple numerical listing of types of supplies used in the different states is shown in Table 1. The first three columns, showing types of wells, form a rough comparison of the depth of the water sources.

Regarding depths—the maximum depths listed are:—

Maine	560 ft
New Hampshire	300 ft
Vermont	450 ft
Massachusetts	300 ft
Rhode Island	750 ft
Connecticut	650 ft

TABLE 1.—TYPES OF WATER SUPPLIES USED.

State	Dug wells and springs	Driven, tubular, gravel-packed	Deep drilled wells	Combined ground sources	Combined with surface
Maine	11	9	7	4	4
New Hampshire	19	9	0	6	7
Vermont	1	11	4	2	11
Massachusetts	31	72	1	37	8
Rhode Island	2	11	6	2	2
Connecticut	9	2	14	9	11
Total	73	114	32	60	43

In recent years a number of gravel-packed wells have been developed, usually to augment other sources, and therefore a comparison of types of new wells developed was made (see Table 2). The oldest gravel-packed well listed is in Portsmouth, N. H., and was constructed in 1922. This date, therefore, was taken as the start for this comparison. New driven-well systems are listed as one, rather than counting the actual number of individual wells. It should also be noted that in almost all cases the new driven-well systems were additions to existing driven-well fields.

TABLE 2.—TYPES OF NEW WELLS DEVELOPED SINCE 1922.

State	Dug or springs	Driven	Deep drilled	Gravel-packed
Maine	3	3	2	6
New Hampshire	14	9	1	13
Vermont	1	3	8	10
Massachusetts	21	73	0	46
Rhode Island	3	8	5	8
Connecticut	6	7	16	16
Total	48	103	32	99

It will be noted that, while deep wells in rock are in the minority, there are a few of these with substantial yields, such as the wells of the Williamsville Water Co. in Killingly, Conn., with a yield of 250

gpm; Hazardville Water Co. of Enfield, Conn., with a yield of 150-200 gpm, and the Harrisville Fire District of Burrillville, R. I., with a yield of 192 gpm.

There are some dug wells with high yields, such as the well of the E. Lyme, Conn., Water Commission with a reported yield of 1,100 gpm; Campton, N. H., 500 gpm; Calais, Me., 970 gpm; Clinton, Me., 500 gpm; Needham, Mass., 520 and 690 gpm; Newton, Mass., 690, 690 and 830 gpm; Wellesley, Mass., 520 gpm, and Attleboro, Mass., (2 wells) 1,040 gpm. Under favorable conditions this type of well may provide an inexpensive water source.

No wells of the horizontal-collector type are listed. Several industrial installations of this type of well have been made in New England and it is understood that at least one municipality is considering the installation of this type of supply.

It is recommended that operators of ground-water supplies keep records of water-table fluctuation and obtain as much information as possible on new well construction. The U. S. Geological Survey is interested in coöperating with operators in increasing its program of periodic measurements of water levels in the New England States.

Your committee feels that the tabulations in this report may provide a background for further studies, which might include the legal status of ground water in New England, recommendations for the construction and protection of various types of ground-water supplies, recommendations concerning yield tests and perhaps more detailed information on the treatment of ground water. A bibliography of reports on ground water in New England might also be attempted.

The earlier work of former members of the committee, Joseph C. Knox of Massachusetts and Leonard W. Trager of New Hampshire, is gratefully acknowledged.

Respectfully submitted,

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William A. Healy
Vincent M. Hynes
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Walter J. Shea
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(*Signed*) Leslie K. Sherman, *Chairman*

SUMMARY OF SYMBOLS UNDER VARIOUS COLUMN HEADINGS IN ACCOMPANYING TABULATIONS

Estimated Population Supplied

W = Winter
S = Summer

Type of Well

D = Dug
T = Driven
A = Deep well in rock
G = Gravel-packed
S = Spring

Well ends in —

S = Sand
G = Gravel
R = Rock
C = Clay

Treatment

(These symbols were taken from "Notes Regarding State Directories of Water Treatment Plant Data," U. S. Public Health Service, Cincinnati, O., May, 1941.)

A = Aeration
Ac = Aeration, coke bed
F = Filtration
Fs = Filtration, slow sand filters
Fr = Filtration, rapid sand filters
Fp = Filtration, pressure filters
Fz = Filtration, zeolite filter
D = Disinfection
Dh = Disinfection, by hypochlorite
Dc = Disinfection, by chlorine
N = Ammoniation
Ng = Ammoniation, with NH_3 gas
Nc = Ammoniation, by ammonium compound
I = Iron or manganese removal
K = Chemical dosage for corrosive correction or water stabilization
Kp = Chemical dosage for corrosive correction or water stabilization, by alkali feed for pH adjustment
Kc = Chemical dosage for corrosive correction or water stabilization, by feed of phosphate compounds
Ko = Chemical dosage for corrosive correction or water stabilization, by sodium silicate

Type of Pump

G = Gravity
C = Centrifugal
D = Displacement
T = Turbine
A = Air-lift
J = Jet

CONNECTICUT PUBLIC GROUND WATER SUPPLIES

Town	Utility	Population Supplied	Present Consumption MGD	Number of Wells	Location	Type of Well[3]	Depth	Diameter	Well ends in[2]	Date of Construction	Test Yield gpm	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment[5]	Type of Pump[4]	Capacity of Pump gpm	Present Daily Pumpage MG	Remarks
Avon	Avon Water Co.	750	.065	1	Near Pond	G & A	108′	12″	R	1934	120)	<0.1	65	7.1	9	—	T	100	.046	
				1	Near Brook	G	30′	24″	G	1945	214)	0.2		6.1	25	—	T			
Bethel	Bethel Water Dept.	4,200		1	Near Pond	G	65′	12″	G	1947	500	<0.1	94	8.0	1.0	Dc		350		
Bridgeport	Bpt. Hyd. Co., Westport Div.	8,000		1		D	15′	30′	G	1892		0.2	58	6.4	20	Dc	C	425	.075	Auxiliary Supply
Bristol	Bristol Water Dept.	35,000		1		G	76′	12″	G	1947	800	<0.1	40	6.5	16	Dh	T	800		Auxiliary Supply
Canaan	Falls Village Lt. & W. Co.	100		1	High Ground	A	210′	6″	R	1932	)	<0.1	284	7.4	20	—	D	4	.002	Flowing Well
				1	" "	S	6′	15′x21′	R		)									
Colchester	Boro of Colchester	1,500	.15	2	Near Brook	G	42′	12″	G	1939)	200	0.1	28	7.5	5	Kp	T	85(throt)	.15	
							39′	12″	G	1939)							T	127)		
Cornwall	Cornwall Water Co.	130		4	High Ground	S	2′					<0.1	98	7.3	10	—	G			
Coventry	Lakeview Terrace Supply Co	300		1		A	426′	8″	R	1941	)	0.1	87	7.1	12		D	10		
				1		A	538′		R	1941	)						D	3		
				1		A	300′		R	1946	3.5						D	3		
				1		A	300′		R	1946	7.5	0.1	98	6.8	25		D	10		
	So. Coventry Water Supply Co.	600		1		A			R	1942	15	3.0	44	6.1	37		J			
				2		A	250′	8″	R	1947	16	0.5	84	6.9	18	—	J			
Cromwell	Cromwell Fire District	3,200	.322	1	Near Stream	G	24′	10″	G	1947)	100)	<0.1	75	7.1	11	—	(C	200	.322	
				3	" "	T	24′	6″	G	1941)	)									
							31′	6″	G		45)						(C	175		
							43′	6″	G	1942	129)									
Durham	Durham Aqueduct Co.	200		1		A	205′	4″	R		)	0.6	62	6.5	15	—	D	15		Auxiliary
						D	14′	15′x24′			)									
East Lyme	East Lyme Water Comm.	500	.015	1	Near Pond	D	35′	6′x16′	S		1,100	0.2	38	6.9	8	—	D	120	.015	
East Windsor	Broad Brook Co.	650	.05	1		A	362′	6″	R	1925	30)	<0.1	102	7.4	5	—	D	25	.05	Wells used alter-
				1		A	220′	6″	R	1925	45)						D	25		
				1		A	310′	8″	R		50)						D	60		nately
Ellington	Ellington Water Co.	350		1		A	255′	6″	R		30	0.1	105	7.6	4.5	—	D			
Enfield	Conn. Lt. & Power Co.	13,600	.733	16		S					870	0.1	71	7.2	6.5	Dc	G			Gravity to pump basins
	Hazardville Water Co.	1,800	.042	2		A	253′	8″	R	1910	)	0.1	86	7.5	3.5	Kp	(T	172	.042	
							259′	8″	R	1918	150-200)						(T			Emergency

CONNECTICUT PUBLIC GROUND WATER SUPPLIES

Town	Utility	Population Supplied	Present Consumption MGD	Number of Wells	Location	Type of Well[1]	Depth	Diameter	Well ends in[2]	Date of Construction	Test Yield gpm	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment[3]	Type of Pump[4]	Capacity of Pump gpm	Present Daily Pumpage MG	Remarks
Farmington	E. Farm. Hts. Water Co.	120		2		A	146′	6″	R	1928)	55	0.1	147	7.4	11	—	D	15		
						A	170′	6″	R	1941)							D	75		
Griswold	Jewett City Water Co.	4,400	.350	1	Swampy Area	G	62′	8″	G	1940	365	0.4	20	6.6	4.5	Dh	T	250	0	Auxiliary Supply
Kent	Kent Water Co.	450	.048	1		A	190′	10″	R		40	0.3	140	7.6	7	—	C	44 or 75		Not in use
				1		G	23′	12″	G		30	0.2	193	7.1	30	—	T			
Killingly	Williamsville Water Co.	1,000	.08	1		A	88′	12″	R	1941	250	<0.1	63	7.0	11	—	T	500	.08	
	William Prym, Inc.	315	.02	1	Near Canal	A	458′	8″	R			<0.1	60	7.2	8	—	D	30	.02	
Litchfield	Litchfield Water Co.	1,300	.25	2	Meadow	G	60′	8″	G	1940	160)	0.1	124	7.3	12	—	T	105		Also Surface Supply
							45′	8″	G	1940	60)						T	105		
				3	Meadow	T	56′	8″	G		)									
							56′	8″	G		)	0.0	130	7.5	8.5	—	2C	100 ea.		
							51′	8″	G		)									
	Bantam Housing Project	320		3		A	400′	8″	R	1942	13	0.4	110	7.3	10	Dh	D			
						A			R							Dh	D			
						A			R							Dh	D			
Manchester	Highland Park Water Co.	150		1		D	15′	36″	G	1910		<0.1	28	6.2	19	—	D	15		
	Manchester Water Co.	6,800		3		G	27-35′	10″	G	1944	125-225 ea.	0.1	28	6.1	22	—	G			Gravity to Reservoir Tributary
	Manchester Water Dept.	23,000	1.2	1		T	60′	10″	G	1925	600	<0.1	36	6.3	22	—	C	600		Combined with surface
Middlebury	Westover Water Co.	400	.03	3	Near Pond	D	18′	18′	G	1931		0.1	32	6.2	19	DhKc	D	84		
				1		S		15′x30′	G		3.5						D	26		
				1		A	650′	8″	R									12		
Naugatuck	Naugatuck Water Co.	16,000	4.5	1		G	76′	20″	G	1944	300	<0.1	40	6.3	18			300		Combined with surface

CONNECTICUT PUBLIC GROUND WATER SUPPLIES

Town	Utility	Population Supplied	Present Consumption MGD	Number of Wells	Location	Type of Well[1]	Depth	Diameter	Well ends in[2]	Date of Construction	Test Yield gpm	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment[3]	Type of Pump[4]	Capacity of Pump gpm	Present Daily Pumpage MG	Remarks
New Britain	New Britain Water Dept.	72,400	8.0	20	Near Canal	T	50′	8″	S & G	1922	1,220	<0.1	39	6.7	6	Dc	D	1,950	2.8	Combined with surface
				71	" "	T	18′-24′	2″	S & G	1944	1,390							700		
New Canaan	New Canaan Water Co.	6,000	0.57	1		A	125′	6″	R	1895		0.3	64	6.7	8		D			Not in use
New Hartford	New Hartford Water Co.	1,000		1		G	82′	6″	S	1945	90	0.0	14	5.8	17					Emergency Supply.
North Canaan	North Canaan Water Co.	1,200	.28	2	Near River	T	40′	10″				—	125	7.3	13	—	D	150	.04	Auxiliary
				2		D	12′	15′x15′				—	122	6.0	High	—	D	150		Supply.
Norwich	Ahern Text. Print Co.	400		1	Near River	A	506′	8″	R		40	0.3	103	6.9	17	Dh	D	40		
Plainfield	Plainfield Corporation	1,500	.105	1		D	8′	40′x12′		1912		0.1	36	6.5	14			600	.105	
	Aldrich Heights Water Co.	230		1		A	212′	6″	R	1935		<0.1	64	7.4	5.5	—	D	35		
	S. H. Trask	165		1		A	76′	6″	R	1935		0.1	88	6.6	20	Dh	T	25		
	Union Water Works Co.	320		1		A	250′	6″	R	1935		0.1	169	7.3	13	Dh	D	25		
	Floyd Cranska	350	.024	2		A	110′	6″	R			0.1	57	7.0	11	Dh	D	18	.024	
Plainville	Plainville Water Co.	7,500	.392	2	Near Pond	G	66′	10″	S	1944	500	0.0	58	6.7	16	—	T	500	.255	Combined with surface
						G	64′	16″	S	1947	800	0.1	98	7.8	3					No pump installed
Plymouth	Terryville Water Co.	3,400		1		G	67′	26″	G	1926	415	0.1	21	6.4	8	—	T	400	.168	Surface auxiliary
Salisbury	Lakeville Water Co.	3,000	.40	4	Salisbury	T	25′	4″	G	1929		0.3	190	7.3	18	—	C	250	.155	Auxiliary
				2	Lakeville	T	35′	10″	S & G	1945	150	0.0	68	7.2	9			85		Supply
Simsbury	Westover Plain Water Co.	125		1		S	6′	12′x20′		1901		0.1	50	7.0	8	—	G			
	Tariffville Fire District	300		1	Near River	G	50′	10″	G	1941	265	0.2	226	7.6	4	—	T	200		
Somers	Philip Leroux	219		1		A	210′	6″	R			0.1	133	7.3	11	—	D	25		
				1		A	65′	6″				<0.1	110	7.3	9	—	J			
Southington	Southington Water Dept.	9,420	1.105	1		G	55′	16″	G	1943	500	0.3	104	7.1	13	Dc	T	450		Used with surface supply
Sterling	U. S. Finishing Co.	450		3		S	8′	15′x15′				0.2	16	5.9	14	—				
							8′	8′x10′												
							10′	10′												
				1		A	180′	10″	R											

CONNECTICUT PUBLIC GROUND WATER SUPPLIES

Town	Utility	Population Supplied	Present Consumption MGD	Number of Wells	Location	Type of Well[1]	Depth	Diameter	Well ends in[2]	Date of Construction	Test Yield gpm	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment[3]	Type of Pump[4]	Capacity of Pump gpm	Present Daily Pumpage MG	Remarks
Suffield	Conn. Lt. & Power Co.	4,300	.175	2		A	230′	8″	R	1895	)	<0.1	191	7.5	8	—	T	190	.175	
				1		A	247′	10″	R	1944	)						T	50		
Thompson	Masonville Spring Water Co		.228	1	Near Stream	D	11′	30′	G	1907		0.2	33	6.5	10	DhKc	C	215	.228	
	Mechanicsville Supply Co.	300		1		A	450′	8″	R	1911	)	<0.1	48	7.1	6	—	D			
						S	Small				)									
				1		D	25′	6′			)									
Vernon	Talcottville Water Co.	200	.003	1		G	54′	10″	G	1948	136	<0.1	82	6.9	17	—	T	100		
				1		D	18′	5′	G		Small						G			
				1		D	12′	10′	G		Small						G			
				1		A	502′	8″	R	1915	50						D			
Washington	Judea Water Co.	400	.035	3		A	125′	8″	R		12	)					D			
							60′	8″	R		40	)	0.2	43	6.3	22	—	D			
							100′	8″	R		10	)						D			
				Several		S	Small			1898		)						G			
	Bryan Memorial	200		1		S	6′	6′x11′		1939								G		
				1		A	137′	8″	R		27		0.1	66	6.5	22	—	D	8		
Waterford	Best View Water Company	150		1	Near Brook	D	12′	15′		1913			0.5	49	6.0	60	Dh	D	21		
Watertown	Watertown Fire District	3,000	.29	3	Near Stream	D	16′	16′	G	1923	49	)									
						D	20′	16′	G	1923	104	)	<0.1	45	7.1	6	Kp	D	300	.29	
						D	22′	16′	G	1923	170	)									

[1] A = Deep Drilled Well
D = Dug Well
G = Gravel Packed Well
T = Tubular or Driven Well
S = Spring

[2] G = Gravel
R = Rock
S = Sand

[3] D = Disinfection
h = hypochlorite
c = chlorine gas

K = Corrosion Control
c = phosphate
p = alkali

[4] A = Air Lift
C = Centrifugal
D = Displacement
G = Gravity
J = Jet
T = Turbine

MAINE PUBLIC GROUND WATER SUPPLIES

City or Town	Utility	Population Supplied	Present Consumption MGD	Number of Wells or Springs	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction	Yield GPM	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump GPM	Present Daily Pumpage MGD	Remarks
Berwick	Berwick Water Dept.	1,220	.189	2 W	Field on edge of town	D	17′	15′ Sq.	G	1901		.3	16	6.2	19.0	Dc	C	350	.189	Flooding possible
Brownville Jct.	Brownville & Williamsburg Water Co.	990	.032	1 W 2 S	In stream gravel bed	D	14′	10′ Sq.	G	1929		1.3	39	6.2	13.0	Dc	D	25	.032	Flooding possible
Brunswick, Topsham	Brunswick & Topsham Water District	10,250	.848	85 W	Open country, 500 yds. from river	T	18′ to 35′	2½″	S	1905		1.0	31	6.7	22.0	Dc Ng	C	800 and 1,000	.848	Flooding possible
Calais	Calais Water & Power Co.	6,920	.573	1 W	Open field, St. Stephen, N. B.	D	20′	30′	G & R	1906	970	.05	39	6.5	11.0	None	2D-1C	1,800 and 1,500	.750	Surface standby
Clinton	Clinton Water Dist.	440		1 W	Open field, edge of town	G	42′	18″	G	1946	500	.1	104	7.4	7.0	DhFz	C	300	.025	
Cornish	Cornish Water Co.	660	.059	4 S	Woods, edge of town	S				1911		.07	32	6.6	7.0	None	T	1,200	.096	
Cumberland	Cumberland Ctr. Water Co.	330	U	1 W 4 S	High wooded ground 4 miles from town	T	100′	2½″	G	1925		.45	40	7.0	8.0	Dh	—	—	—	
Danforth	Danforth Water Dist.	490	U	5 W	Hillside, 1.5 miles from town	A	150′ to 350′	4″	R	1931		.35	80	7.1		Dh	D	40	—	
East Millinocket	Northern Water Co.	960	.250	1 W	Gravel pit, 1,000 ft. from river	G	22′	24″	S	1940	800	.12	47	7.2		Dc	C	350	.250	Surface standby
Friendship	Friendship Water Co.	385	U	4 W	1 in town—3 out in wooded area	D	—	—	G	1900		.23	22	6.1	5.0	None	D	100	.005	
Harrington	Quantabacook Water Co.	445	.006 (E)	1 S	Wooded area, 1.5 miles from town	—	8′	15′x25′	S	1911		.07	31	6.6	7.0	None	None	—	Gravity	

MAINE PUBLIC GROUND WATER SUPPLIES

City or Town	Utility	Population Supplied	Present Consumption MGD	Number of Wells or Springs	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction	Yield Tests gpm	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump GPM	Present Daily Pumpage MGD	Remarks
Houlton	Houlton Water Co.	5,850	.660	1 W	Glacial drift outside town	G	40′	30″	G	1940	1,040	.07	150	7.6		Dc Ng	C	2 @ 700	.660	Part surface in summer
Kezar Falls	Kezar Falls Water Co.	1,095	.064	1 W	Edge of town	T	240′	8″	G	1909	200	.33	17	7.3	5.0	Kp	1C-1D	160	.064	
Limerick	Central Maine Power Co.	450	.010	2 W	Springs on hillside		12′	10′x14′	C	1925										
				5 S	Wells, edge of town	D,A	560′	10″	R	1913		.30	33	6.3	20.0	Dh	1C-1D	42	.010	
Lubec	Lubec Water Dept.	2,060	.179	1 W	¼ mile off road 2 miles from town	D	24′	10′	G	1931	170	.05	108	7.6	2.0	Dh Nc	C	500	.179	
Milbridge	Milbridge Water Co.	435	U	2 W	Wooded area 1		25′	18′		1900										
				2 S	mile from town	D,T	100′	4″	G	1916		.08	24	6.3	16.0	None	D		.013	
Monson	Monson Spring Water Co	460	U	3 W	Wooded area, ½ mile from town	A	315′ 272′ 115′	6″	R	1937		1.2	20	6.8	5.0	None	D	50	.015	
New Sharon	New Sharon Water Co.	110	.014	1 W	Open field—hillside															
				1 S	1 mile from town	A	53′	6″	R	1911	6	T	51	7.2	0	none	C	5	.007	
North Bridgton	North Bridgton Water Co. No.	285	U	2 S	Wooded area, hill above village	—	—	—				T	25	7.1	4.0	None	Gravity		U	
North Jay	North Jay Water Dist.			1 W	Open field near town.	New utility—No information yet available														Flooding possible
No. New Portland	No. Village Water Co.	215	U	2 S	Woods, several miles from town	—	—	—	—	——		.07	21	6.7	4.0	None	D	15	U	
Paris	Paris Hill Water Co.	225	U	2 W	Wells in village, springs		440′			1925		.15	18	6.7	5.0	None	D	15	U	
				12 S	on hill mile outside	A	270′	6″	R	1930										
Patten	Patten Water District	265	.008 (E)	1 W	In village	New well—No information yet available														

MAINE PUBLIC GROUND WATER SUPPLIES

City or Town	Utility	Population Supplied	Present Consumption MGD	Number of Wells or Springs	Location	Type of Well	Depth	Diameter	Well ends in	Date of Construction	Yield Tests gpm	Analyses: Iron	Hardness	pH	CO_2	Treatment	Type of Pump	Capacity of Pump GPM	Present Daily Pumpage MGD	Remarks
Pittsfield	Pittsfield Water Works	2,490	.700	4 W	Woods—River bank 1 mile from town	D	3-25′ 1-20′	18′x18′ 53′ d.	S,G	1894		T	32	6.5	11.0	Dc	1D-1C	300	.700	Flooding possible
Port Clyde	St. George Water Co.	275	.032	2 W	Edge of village	A	200′	6″	R	1911		.28	47	7.1	6.0	None	D	25	.032	
Rumford (section of)	Smith & Greene Water Co.	105	U	2 W	Hillside above populated area	D	6′	50′x100′ 10′x15′	G R	1908 1940		.05	20	6.8	3.0	None	Gravity		U	90% surface
Sanford	Sanford Water District	14,700	.825	63 W	Wooded area 1.5 miles from town	T	10′ to 30′	8″ to 2½′	G	1902 to 1934	50-100	.26	22	6.8	20.5	Fz Dc,Ng	C	1-350 1-700	.825	Flooding possible
Skowhegan (section of)	W. Skowhegan Aqueduct Co.	710	.024 (E)	3 S	Flat sandy area populated	D	6′	1-18′ 2-6′	C	Unknown		T	40	6.3	7.0	None	Gravity			
South Berwick	S. Berwick Water Co., Inc.	1,520	.140	1 W	Wooded area 1 mile from town	G	53′	12″	R	1937	100	.25	68	6.5	21.0	A,Fr,Dh	T	200	.140	
South Paris	So. Paris Village Corp. W. W.	2,330	.763	2 W	Glacial drift on edge of town	G	47′	24″	G	1937	685 712	T	26	6.3		None	T	2-500	.763	
Stratton	Stratton Water Works	370	U	3 S	Wooded hillside 3 miles from town	—	—	—	—	——		.06	14	6.6	3.0	Dh	Gravity			50% surface
Union	Union Water Co.	265	U	1 W	Well used dry seasons only. Open	A	140′	6″	R	1900		.80	12	6.6	8.0	None		50		
Waldoboro	Waldoboro Water Co.	700	.048	4 W	Open country 2 miles from town	A	3-100′ 1-200′	8″	R	1900		.09	32	6.5	5.0	None	D	45	.048	
Warren	Warren Water Supply Co	660	.043	2 W	Wooded pasture 1.5 miles from town	A	1-200′ 1-125′	6″	R	Unknown		.90	41	7.1	6.0	None	D	45	.043	
Washburn	Washburn Water Co.	650	.053	3 W	Field, edge of town 100 ft. from stream	T	34′ to 38′	6″	G	1916	675	0	225	7.5	4.0	None	D	500 & 175	.053	
Yarmouth	Yarmouth Water Dist.	3,020	.183	1 W	Pasture. 3 miles from town	G	46′	30″	G	1947			30	7.5		Dh	T		.183	

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Adams	Fire Dist.	13,000	0.269	6	Near Hoosick River	T	106	6"	C	1902	2.5	1902	.04	108	8.0	1.0	Dc	D	2,990		Can be flooded.
Amesbury	Municipal	9,000) W								1910											
	Rings Corner	9,500) S	0.746	120	Powow River	T	30-60	2½"		1939	0.875	1940	5.3	59	6.8	27	AcFs	D	700		
	Market St.	50		1	¾ Mile E. of L. Gardner	D	45	18		1884			.17	140	7.7	9	None	D	50		6" Tubular Well 200 ft. deep in bottom of dug well.
Ashland	Municipal	2,900	0.683	12	Fleasant St.	T	28	2½"	G	1911			.28	36	6.0	—		D	150		
				31	High St.	T	30	2½"	G	1919-38			.33	25	6.1			D	500		
				1	High St	D	33		G									D			
				1	Cedar St	G	64	12"	G, L	1941	0.72	1941	.03	67	7.2			T	500		
				1	Cedar St	G	42	12"	G, L	1942	0.72	1941						T	500		
Attleboro	Municipal	22,000	1.230	1	South Attleboro	D	25	30	S, G	1892)	1.500	)	.06	39	7.3	2.2	)	C			Gravel Packed wells at So. Attleboro are treated. Analyses reflect this treatment.
				1	" "	D	40	25	G	1904)		)					)				
				1	Mechanic St.	G	35	18"	G	1935	Abandoned										
				1	Bank St.	G	35	18"	L	1935	Abandoned							D			
				88	W. Mansfield	T	18-30	2½"	G	1929	Abandoned										
				3	W. Mansfield	G	30	6"	G	1935	0.432	1935	.19	17	6.0	11		T, C	500		Aux. supply.
				2	South Attleboro	G	35	18"	G	1936	0.905	1936	.05	25	6.0	18	A, Kp	T, C	500		Used with Dug Wells.
Auburn	Water Co.	3,000			Banks of Ramshorn Brook	T	26	2½"		1919	0.220		.06	70	6.8	41	None	D	30		
Avon	Municipal	2,500	0.1367	1		D	22	20	G, L	1889	0.070	1889	.05	35	5.8		None	D	500		
				1		A	201	8"	L	1889	0.094	1889						D	60		
				1		A	225	8"	L	1889	0.065	1889						D	60		
				1		G															
Ayer	Municipal	3,500	0.191	1	West Shore of Grove Pond	D	18	25.0'		1887	0.240		.27	21	6.1	25	None	D	380		Test Yield Estimated.
				45	" " " " "	T	34	2½"		1906 1927	0.400		.67	38	6.5		None	D	380		Test Yield Estimated.
Barnstable	Water Co.	4,000 W	0.529	19	Upper Gate Pond &	T	25-35	24"	S	1912	1.152	1912)	.25	13	5.6	18	Kp	D	800		
		8,500 S		32	Mary Dunn Road	T	30	2½"	S	1925	0.690	1925)									
				33	Cedar Swamp	T	30	2½"	S, G	1926	0.850	1926	.10	8	5.9	17.6	None	D	800		Auxiliary supply.
	Centerville-Osterville Fire District	2,109 W 5,000 S		50	McShane Property	T	45-50	2½"	S	1938	0.720	1938	.22	6	5.7	9.2	None	C	250		
				3	Arena	G	60	8"	S	1938	0.432	1938			5.9	13	None	C	250		
	Cotuit Fire Dist.	900 W 5,000 S	0.060	4	Lovell Pond	G	30	6"	S	1936	0.720	1936	.11	7	5.4	15	None	C			
Bedford	Municipal	2,100 W	0.145	1	North of Kendrick Pond	D	21	20	G	1908	0.060		.09	14	6.1	11	None	D	250		
				1	E Bank of Shawsheen R	T	30-35	2½"	G	1925	0.360		.03	19	6.2	31	None	D	250		
				1	" " "	D	17	4½	G	1939											

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Belchertown	Water Dist.	600	0.032	9	West of Jabish Brook	T	32	2½"		1925	0.150		.04	31	6.3	15	None	D	200	.028	
Bellingham	Municipal	3,500	0.114	1		G	41	48"	G	1939	0.460		.05	16	6.4		None	T			
				1		G															
Bernardston	Fire & Water Dist.			1	East of Mill Brook	D	30	12.5	S & G	1936			.07	17	7.6	2.5	None	T	100		
Billerica	Municipal	4,000 W	0.324	1	W. Bank of Concord R. (Well #1)	G	37	18"	G	1932	2.000		1.04	27	6.1	28	A, Dc	T	350		
		6,000 S		1	W. Bank of Concord R. (Well #2)	G	38	18"	G	1932	2.000		.76	28	6.3	.15	A, Dc	T	350		
Bourne	Water District	1,000 W	0.081	3	East of County Road	G	49-71	18"	S	1939	1.152	1939	.05	8	5.8	11	None	C	800		
		5,000 S																			
	Buzzards Bay W. D.	650	0.053	25	Bournedale Road	T	32-68	2½"	S	1937	0.700	1937	.04	6	5.7	11	None	C	275		
	Sagamore Beach Development Co.			9	Near Sagamore Beach	T		2"	S	1928			.04	14	5.7	20	None				
	Ware Tenement	——	——	4	North of Tenement	T	58	2½"	S	1914			.11	16	5.8	13	None				
Bridgewater	Municipal	5,000	0.021	31	High St. at Matfield River	T	28-40	2½"	S, G	1915,25			.05	20	5.7	19	None	D	450		Wells flooded occasionally
Brookfield	Municipal			—	Regular Supply is Spring-Fed Surface Reservoir																
				3	Banks of Quaboag River	T	12	3'-4'		1931	——		.12	31	6.0)	—	Kp	D	75		
				15	Banks of Quaboag River	T	12	2½"		1931	0.036				)		Kp	D	75		
Brookline	Municipal	57,000	5.197	175	Charles R.	T	40-90	2½"	G	1889-94			1.45	44	6.5			D	3,470		
				4	" "	G												T	2,780		
Canton	Municipal	6,700		19	Springdale	T	20-40	2½"	G	1893							None	C			
				1	Springdale	D	25	40	G	1889	0.200		.07	24	6.4		None	C	700		
				1	Henry's Spring	D	24	40	G	1894			.05	32	6.0		None	C	400		
				1	Ward Well	D	25	35	G		0.275		.16	28	6.4		Dc	C	500		
Chatham	Water Co.	1,800	0.108	38	Chatham-Harwich Line	T	29-36	2½"	S	1933,35			.03	8	5.8	13.5	None	C	500		Regular Supply.
					White Pond	Filter Gallery				1929 1913			.06	12	—	4.8	None				Emergency Supply.
Chelmsford	Water District N. Chelmsford F. D.	3,024	0.185	25	River Meadow Brook	T	40	2½"	G	1940	1.000		.06	31	6.2	23	None	D	310		
		3,000	0.103	30	Newfield's Pond	T	35	2½"	G	1907			.87	24	6.3	26	None	D	220	.103	
Cohasset	Water Co.	3,500	0.348	62	Near R. R. Sta.	T	12-40	2"	G, L	1886	0.750		Not used								
				7	Elms Meadow	T	20-38	2½"	G, L	1887	0.220		.03	50	6.5			D	150		
				1	King St. at Lily Pond	Filter Gallery 200x8x12			G	1902			.65	11	6.0			D	175		
				1	Beechwood	D	25	25		1908			1.6	17	5.7		Ac.DcFr	C	550		

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Colrain—																					
Griswoldville	Kendall Co.	325	0.030	1		S & W	291	10"					.05	58	7.0	4.0	None				
Lyonsville	Kendall Co.	143		1		S & W							.18	37	6.8	6.0	None				
Cummington	Municipal	210	0.005	4	1½ Miles S. W. of Village	S			R	1930	.043		.13	39	7.2	7.5	None	G			
Dartmouth	Bay View Improvement & Water Co.	100		1	Bay View Colony	T	130	30"	L				.15	49	6.1	15	None	D	35		Close to Houses.
	Salters Point Water Co.	300 S		1	West of Salters Point	D	18	16	L	1898			.05	33	5.9	23	None	D	75		
				1	West of Salters Point	D	12	15	L	1922			.03	46	—	26		D	30		
Dedham	Water Co.	16,700	1.404	35	Charles R.	T	20-35	4"	S, G	1897-1914		)									
				8		T	35-40	8"		1915, 36		)	.14	37	6.2		De	D	1,670		
				1		T	67	16"	S, G	1928		)									
				3		D	18, 30, 30	26, 8, 6	S, G	1881, 1913			.05	41	6.2		"	"			
				1		G			S, G				.15	44	6.4		"	"			
				1		G	60	5	S, G	1928							"	"			
Deerfield	Fire Dist.	700	0.005	8	Near Deerfield R.	T	16	4"	S	1930		)	.28	25	6.6	7.0	None	D)	120		
				4	Scattered	S				1911		)					None	D)			
Douglas	Municipal	2,000	0.281	7	Bad Luck Pond Brook	T	32	2½"		1909	0.400		.08	22	6.2	18	None	D	360		
Dover	Municipal	1,600	——	2	Church Street	T	34, 44	6"	G	1927			.17	46	6.6		None	D	50		
Dracut	Municipal	4,600	0.240	16	Hopkins Street	D	20-30	4	G	1922-28)	0.290		.15	62	6.8	24	Dc	D	250)	.230	
				15		T	20-26	2½"	G	1905)									)		
				24	Lakeview, near Long Pond	T	24-28	2½"	G	1937	.173						Kp	C	125	.070	
Dudley	Municipal	4,000	0.177	38	Merion Pond	T	35	2½"		1900 1929 1936	0.190		.03	16	6.0	19	None	C	400		
Dunstable	Municipal	130	0.004	6	Black Pond	T	25	2½"		1930-36			.38	25	6.3	15	None	D	75		
				1	Austin Pond	D	30	12		1917	0.017						None	D	75		Emergency Supply
Duxbury	Municipal	2,400 W	0.272	22	Ford's Pond (Old Wells)	T	30-50	2½"			.300	)	.04	11	6.0		None	D	350		
		4,500 S		22	Ford's Pond (New Wells)	T	30-50	2½"			.410	)					None	D	550		
East Brookfield	Municipal	900	0.068	1	Westerly Shore Lake Lashaway	D	15'	?		1917			.04	10	6.0	—	Dc	D	100		
Easthampton	Municipal	10,300	0.830	37	1½ miles S. W. of village	T	40	2½"	G	1908) 1924)	0.814		.04	37	7.9	0.0	None	D	2,000		

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Easton	N. Easton Village District		0.265	1	Lincoln Street	D	27	29	S, G	1887			.07	18	6.0		None	C	450		Filtered water from reservoir also enters well. Spring feeds dug well.
				1	Lincoln Street	S															
Edgartown	Water Co.	1,500 W 5,000 S	0.154	1	Great Pond	D	19	15	S	1906	0.360	1906	.11	6	5.5	9.5	None	D	300		
				17	" "	T	35	2½"	S	1927			.03	5	5.5	9.5	"	D	300		
Fairhaven	Water Co.	10,000	0.528	36	Nasketucket River	T	25-35	2½"	G, L	1893			.30	30	5.6	42	None	D	800		
				31	Mattapoisett River	T	30	2½"	G	1901			.09	27	5.6	21	Kc	D	700		
Foxborough	Municipal	6,500	0.932	24	Near Neponset Reservoir (Old)	T	23-50	2"		1892	0.700	1892	.43	19	6.3		None				
				23	Near Neponset Reservoir (New)	T	28-50	2½"		1922							None	C	900		
Framingham	Municipal	25,500	2.315	1	North of Lake Cochituate	G	69	8"	S, G	1939	1.500		.04	66	7.0		"	C	700		
				1	" "	G	69	8"	S, G	1939	1.500		.35	53	6.8		"	C	700		
				1	" "	G	78	8"	S, G	1939	1.500		.04	49	6.9		"	C	700		
Franklin	Municipal	7,500	0.557	12	Beaver Pond	T	22-23	24"	G	1924	)						None				
				6	" "	T	22	30"	G	1925	)		.10	23	6.0			D	1,000		
				8	" "	T	22	2½"	G	1911	(Not in use)										
				1	" "	D	22	20	G	1906	0.500		.42	20	6.2						
				1	" "	G			G	1944			.10	26	5.9						
				1	" "	G			G	1945			.02	13	5.9						
Georgetown	Municipal	1,300	0.038	19	Parker River	T	26-34	2½"		1917 1934			.20	25	6.4	22	Ac	C	113		
Gill	Riverside Water Co.	350			Riverside	S				1888			.12	38	6.6	4.5	None	G			
Gosnold	Mrs. W. H. Allen	130		1	Cuttyhunk	D	35	7	G	1890	0.017	1890	.06	104	5.9	—	None	D	—		
Grafton	Water Co.	2,200	0.157	1	Quinsigamond River	D	17	10		1887			.06	43	6.2	25	None	C	300		
				1	" "	D	28	25		1910							None	C	300		
				1	Quinsigamond River T-ibutary	G	36	16"		1930	0.216		.04	40	6.0	—	None	C	300		
	Fisher Mfg. Co	350	0.017	2	Leland Hill	S							.04	21	6.6	12	None	G	—		
	" "			2	Blackstone River	T	200-300	8"									None	D			Emergency Supply.
	Saundersville W. S.	150	0.007	5	Leland Hill	S	14	Varies					.19	16	6.3	5	None	G			
				1	Blackstone River	D	10	10								37	None	D	30		Emergency Supply.
Granville	Center Water Co.	65		3	¾ Mile W. of Village	S			S	1910			.07	9	6.3	5.0	None	G			
Great Barrington	Fire Dist.	3,000 W 3,500 S	0.707	1	Near Green River	Infilt. Gallery			G	1934	1.15		.06	66	7.3	5.5	Dc	C	1,420		

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Greenfield	Municipal	15,700	1.379	1	Near Green River	D	27	40	G	1913			.03	39	7.1	1.5	Fs, Dc		2,080		
Groton	Water Co	1,460	0.097	1	Braddacook Pond	D	19	30		1897			.23	42	6.9	6	Kp	D	450		
	Water Co. (West)	600	0.232	8	Squannacook River	T	26	2½"		1911			.15	32	6.7	21	None	C	100		
Hamilton	Municipal	1,489 W 1,600 S	0.063	6	½ Mile S. E. of Milas R.	T	26-33	2½	G	1939)	0.324		.07	33	6.3	25	None)	C	225)	.060	6 Tubular Wells in Dug Well to an Additional Depth of 28 ft.
				1	" "	D	16	15	C	1910)							None)		)		
				1	" "	G	32.3	6"	G	1939)							None)		)		
				1	" "	G	49.5	6"	G	1939)							None).		)		
Hanover	Municipal	3,000	0.328	24		T	38-48	2½"	S, G	1930	.252		2.6	38	5.9			C	500		
				1		G				1943	1.008		.59	29	6.0			T	500		
Hardwick	Center	80	0.005	1	½ Mile North of Village	D	10	24x12		1913			.20	21	7.0	—	None	G			
	Gilbertville	1,325	0.080	1	⅛ Mile West of Mill	D	14.5	21		1910			.05	23	7.0	—	None	G			
				2	⅛ Mile West of Mill	D	14.5	39		1922							None	G			
				4	200 ft. from Dug Wells	T	128-197	6"		1911							None	D	22-42		
	Wheelwright	235	0.014	4	Near Hardwick Station	T	31	2½"		1904) 1908) 1924)			.05	12	6.0	—	None	D	28		
Harwich	Municipal	650 W 1,500 S	0.056	4	North of Chatham Road	G	32-38	6"	S	1936	1.000	1936	.06	12	5.8	14	None	C	500		
Hingham	Water Co.	9,700	1.664	1	Free Street	G							.03	50	6.7						Secondary supply.
Holliston	Water Co.	3,300	0.143	1	Winthrop Pond	G	45	6"	S, G	1937	0.282		.07	25	6.4		None	C	250		
Hopkinton	Municipal	2,900		26	Fruit	T	27-41	2½"	G	1929	0.600		.03	22	6.4		None	D	316		
Huntington	Fire Dist.	1,000		9	Near Westfield River	T	22	3"	G	1908	0.288		.05	31	6.2	8.0	None	D	100	None	Used occasionally
Kingston	Municipal	2,777 W 3,000 S	0.266	34	Soules Pond	T	26-41	2½"	G	1897-1922-1940	0.370	1941	.03	13	5.8	16	None	D	300		
Lanesborough	Village Fire & Water District	560	0.021	15	Near Housatonic River	T	31	2½"		1938	0.13		.10	186	7.4	5.0	None	C	150		
Leicester	W. S. District	1,200	0.100	1	Pierce Meadow	D	14	30		1890	0.023		.10	26	6.3	13	None	G			
				5	Paxton	D	14	6		1891							None	G			
				1	"	D	18	6		1903							None	G			
				1	"	T	150	4¼"		1905							None	D	70		
	Cherry Valley & Rochdale Water District	2,300	0.180	2	Henshaw Pond	D	12	30		1910	0.130		.16	19	6.6	6	None	D			Supplement to Filtered Water Supply.

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Hardness	pH	CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
		1,650 W																			
Littleton	Municipal	1,850 S	0.089	9	Beaver Brook	T	20	2½"		1912	0.400		.03	20	6.3	15	None	D	210		
Lowell	Municipal	101,400	5.597	620	N. Bank Merrimack R.	T	24-46	2½"	G				4.4	29	6.5	22	AcFsDc	D			Boulevard Wells.
				30	" " " "	G	35	6"	G	1935											
				210	River Meadow Brook	G							.42	51	6.6		AcFsDc	C			Cook Wells
	Center W.																				
Lynnfield	Dist.	750				T		2½"		1940			.03	84	7.1	5	None	C			
Manchester	Municipal	2,600 W	0.294	1	Saw Mill River	D	29	32"	G	1892			.93	28	6.8	33	Kp	D	350	.360	
		4,000 S		25	" " "	T	27-65	3",4"		1892							Kp	D	350		
										1901											
Mansfield	Municipal	7,000	0.944	1	Pecuanticiot River	D	20	30	G	1888			.04	19	6.0	19	None	D	1,390		
Marblehead	Municipal	11,000 W	0.816	1	Logg's Hill	D	33.5	25		1912	0.300		1.11	68	6.5	27	A, Fs	C	250		Well #2.
		15,000 S		16	Thompson's Meadow	T	34-73	2½"		1923	0.200		1.43	105	7.1	21	Fs	C	120		
Marion	Municipal	1,867 W	0.211	22	Benson Brook (Old P.S.)	T	20-30	2½"	S, G, L	1908-34	0.300	1941	.03	15	5.6	21	None	D	350		
		3,734 S		25	" " " (New P.S.)	T	31-39	2½"	G	1915-38	0.425	1941	.08	38	6.4	16	None	D	350		Wells occasionally flooded
Marshfield	Municipal	2,400	0.433	60	Main P.S.	T	20-36	2½"	G	1927, 30	0.400		.04	10	6.4			D	500		
				7	Humarock Beach P.S.	T	22	2½"	G	1900-20			1.10	19	5.6			D	250		
				1	Brant Rock P.S.	D	19	22	G	1927			.07	12	5.9			D	500		*
				1	Parsonage P.S.	G							.50	40	5.9						
Matta-poisett	Municipal	1,600	0.100	24	Mattapoisett R. (Old P.S.)	T	24-28	2½"	S, L	1913, 23	0.190	1923	.24	26	5.7	27					**
				24	" " (New P.S.)	T	28-35	2½"	S	1924	0.300	1924	.05	23	6.0	23					
Medfield	Municipal	4,200	0.115	63	State Hospital (Old)	T	31-35	2½"		1932	1.440		.04	41	7.3			C	900		
					(New)	T							.24	34	6.3						
Medway	Municipal	3,400	0.128	20	Charles R.	T	18-20	30"	G	1916-39		)	.04	43	6.7			D	350		
				1	Charles R.	G	61	24"	G	1942	0.892	1942)									
Merrimac	Municipal	2,200 W	0.172	32		T	23-46	2½"		1904			.37	30	6.1	30	None	C	558	.172	
		2,300 S								1916											
										1930											
Methuen	Municipal	21,880	1.094	1	Harris Brook	G	47	18"	G	1934	0.400		1.2	35	6.2	33	Dc	T	280	.245	Floods Annually.
				34	Merrimack R.	T	18-34	2½"	G	1922			.05	43	6.1	—	Dc	D	280	.121	Pine Island Sta.
				1	Merrimack R.	G	54	16"	G	1939			.04	35	6.3	25	Dc	T	150	.175	Lone Tree Hill Well #2.

*Used in July & August.

**Wells flooded in Spring.

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Methuen (Cont.)				1	Merrimack R.	G	93	16"	A	1930	.432		1.7	48	6.6	21	De	T	300	.430	Lone Tree Hill Well $3.
				1	Merrimack R.	G	88	16"	A	1930	.288		1.18	48	6.7	24	De	T	200	.225	Lone Tree Hill Well $4.
				1	Merrimack R.	G	88	16"	A	1930	.288		.28	47	6.7	23	De	T	200	.270	Lone Tree Hill Well $5.
Middleboro	Municipal	9,032		1	Nemasket R.	D	22	26					2.9	25	5.8	45	Ac	C	900	.310	
										1885	0.450		.14	25	6.3	8					
Milford	Water Co.	18,366	0.805	2	Charles River	D	25-28	14.5-22			0.400		.27	30	7.7		Kp, De	C	1,390		
				1	" "	Filter															
Millbury	Water Co. (Supplementary Supply in Grafton)	4,350		1	Millbury Ave.	D	35	20		1894	0.100		.04	19	5.8	29	None	C	900		
				1	Quinsigamond R. Tributary	G	36	16"		1930	0.216		.03	26	6.0	—	None	C	300		
Millis	Municipal	2,300	0.253	1		D	12	8x16	G	1892		)									
				1		D	9	24	G	1895		)	.06	57	6.6		None	C	450		
				1		D	10	24	G	1922		)									
				1		D	23	23	G	1930		)									
Monson	Municipal	2,700		2	Near Ingalls Brook	D	25	75		1894	0.400		.04	12	6.4	3.0	Dc	G			
Montague	Bartlett	480	0.024		½ Mile S. of Village	S	4	30"	G	1940	0.079		.07	28	6.9	4.8	None	D			Unprotected against pollution.
Nantucket	Siasconset Water Co.	350 W 2,500 S	0.069	4	Siasconset	D	28	15	S, G	1924-36			.03	19	6.1	16	None	D	160		
	Wannacomet Water Co.	2,900 W 13,325 S	0.047	121	Wyers Valley	T	26-34	2"	S	1911-35		)	.03	21	6.2	11	None	D	500		
				4	" "	D	20-22	8	S	1924-26		)									
Natick	Municipal	15,800	1.196	1	Worcester Street	D	25	30	S, G, L	1902	4.0		.03	54	7.1			C	2,780		
				1	" "	G	63														
Needham	Municipal	14,500	1.038	1	Dedham Ave.	D	24	22	G	1890	0.750		.04	42	6.0			D	1,000		
				1	" "	D	30	20		1900	1.000		.04	39	6.2						
				23	" "	T	10-20	2½"		1914-24	.750		.09	41	6.3						
				38	Great Plain Ave.	T	25-30	2½"		1924-26			.25	38	6.5						
				1	Charles River St.	G							.04	50	6.9			T			
				1	" " "	G							.04	44	6.7			T			
Newburyport	Municipal	15,000	1.569	26	Trout Brook	T	87-252	6"		1904											Also Surface Supply
										1908							Fs	None	——		
				3	Jackman Ravine	D	15			1890	.100						Fs	None	——		
						D	15			1890	.100						Fs	None	——		
						D	40			1895	.200					17	Fs	None	——		
				2	Valley of Trout Brook	S				——							Fs	None	——		

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Newton	Municipal	77,300	6.108	1	Charles River	Infiltr. Gallery	4	4,400x4	G	1885-89)	1.000						Dc, Ng				
				300-400	" "	T	40-70	2½"	G	1889)											
				1	" "	D	38	28	G	1911	1.000		.06	45	6.5		"	C			
				1	" "	D	42	33	G	1927	1.200		.04	36	6.5		"	C			
				1	" "	D	41	20	G	1938	1.200		.04	33	6.5		"	C			
				1	" "	D	32	20	G	1938	.560		.04	41	6.4		"	C			
North Attleborough	Municipal	11,600	0.972	1	North Attleborough	D	29	25	G	1884	.400	)	.12	47	6.9		Kp	C	750)	1,500	
				1	Plainville	D	30	28	G	1905	.750	)						C	750)		
Northbridge	Whitin Mach. Wks.	7,000	0.745	20	Sutton Wells	T	18	2"		1919			.03	7	6.0	19	None	C	850		
				32	Meadow Pond	T	25	2"		1922			.08	24	6.1	26	None	C	600		
	Whitin Mfg. Co.	2,000	0.160	12	Blackstone River	T	33	2½"		1932			.05	25	5.9	28	None	D	110		
Norton	Municipal	2,000	0.203	22	Near Three Mile River	T	30-40	2½"	S, G	1912 on	0.300	1912	.18	19	6.0	19	None	D, C	250		
Norwood	Municipal	16,500	1.404	133	Ellis Station	T	28-45	2½"	S, G	1900-21	1.000		.11	40	6.7		A, Fs, Kp	D	1,040		
Oak Bluffs	Cottage City Water Co.	1,250 W 1,100 S	0.136		Lagoon Pond	*				1890, 1910			.27	11	6.0	10.7	Kp	C	500		*Tile collecting system of 2,200 ft. of 6", 8", 12" pipe.
Oxford	Water Co.	3,200	0.167	8	French River Tributary	T	28	2½"		1906			.06	24	6.3	16	None	C	300		
				7	French River Tributary	T				1914											
				10	French River Tributary	T				1925											
Palmer	Fire Dist. #1	2,800	0.178	8	Near Quaboag River	T	30	2½"	S & G	1922	0.25		—	—	—	—	None	D	224		Not used. Can be flooded.
	Bondsville Water Co.	1,181	0.182	10	Near Quaboag River	T	37	2½"	G	1908			.10	30	6.2	13	None	D	100		
	Cheney	1,700		1	Near Quaboag River	T & S	80	8"	G	1940	0.079		.04	16	6.6	2.0	None	D			
Paxton	Municipal	285	0.014	1	Shore of Asnebumskit Pond	S	7	40		1936	0.145		.20	12	6.6	10	None	C	300		
Pepperell	Municipal	2,000	0.258	34	Banks of Gulf Brook	T	32	2½"		1909	0.350		.07	16	6.1	15	None	D	600		
Provincetown	Municipal	3,668	0.406	50	North Truro	T	21-41	2½"	S	1907-28	0.650	1938	.05	44	6.1	13		D	690		**
				3	Provincetown	G					Raw—		2.3	18	5.3	27	Ac				Auxiliary supply.
											Treated		.37	21	6.5	3					
Reading	Municipal	10,900	0.671	76	Ipswich River	T	38	2½"		1930			1.0	45	6.1	43	AcFs	C			

**Salt water problem, 1932.

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Rockport	Municipal	3,556	0.349	23	Mill Brook Valley	T	21-37	2½"	G	1926) 1938)	.200		.04	23	6.1	14	Kp	C	300	.129	Also Surface Supply.
Russell	Municipal	1,000		1	Near Westfield River	G	30	12"	G	1932			.03	21	6.3	8.5	None	C		None	Not used.
Salisbury	W. S. Co.	2,500 W 6,000 S	0.282	2	Chase's Brook	D	40	22	G	1915)	.432						None	D	400)	.282	So. Well.
							41	16	G	1920)							None	T	700)		No. Well
				1	Chase's Brook	G	45	18"	G	1930)							None		)		Inside of No. Well
Scituate	Municipal	4,900	0.702	35	Beaver Meadow	T	35-45	2½"	S, G	1900-04			.31	44	6.6						
					Webster																
				16	Meadow	T	44-66	2½"	S, G												
				1	Bound Brook	G	21	18"	G												
				1	Kent Street								.08	27	6.5						
					Stearns Meadow																
				1	Well #17								.04	25	6.6						
					Beaver Hole																
Sharon	Municipal	4,100	0.4271	40	Meadow Brook	T	19-50	2½"	S	1906, 1923	.288		.90	26	6.5		None	C	580		New Supply.
				1	" " "	D	16	16	S	1885)	.240	)	.04	60	6.7		None	D	) 500		Old Supply.
				8	" " "	T	21-30	2½"	S	1901, 1911)		)							)		
Sheffield	Water Co.	895		3	East of Town	S				1897 1934			.03	18	6.5	6.5	None	C			
	Shirley Village W. D.				Near Catecunemaug			50													
Shirley		1,975	0.097	1	Brook	D	10	8		1903			.04	37	6.0	21	None	D	168		
				4	" "	D	15	18.5		1909 1910 1914							None	D	168		
Shrewsbury	Municipal	7,300		2	South Street	G	30	3		1933	0.456		.05	25	5.8	23	None	D	320		
				3	Oak Street	G	27	3		1934	0.648		.05	31	6.0	19	None	D	650		
Somerset	Municipal	5,000	0.263	45	Brook St., Dighton	T	28-35	2½"	S, G	1926, 27	0.500	1927	.65	26	7.5	0	Kp	D	450		Treated water analyses.
South Hadley	Fire Dist. #2			2	Elmer Brook	D	12	25	G	1911		)	.09	32	6.9	5.0	None	C	300		
							12	30	G	1911		)						D	300		
Sterling	Municipal	425	0.022	12	Worcester Rd.	T	25	2½"		1934	0.173		.06	36	6.7	26	None	C	110		
	Hill Water	20 W																			
Stockbridge	Dist.	180 S		2	Rattlesnake Mt.	S				1860	0.046		.05	44	7.3	2.3	None	G			
					Brook Below																
Sturbridge	Municipal	850	0.069	15	Cedar Pond	T	25	2½"		1935			1.15	25	6.4	—	None	C	150		
Sudbury	Municipal	2,000	0.033	15	Hop Brook	T	34-62	2½"	S, G	1936	0.145		.04	26	6.9		None	C			
Sutton	Water Co.	250	0.015	1	Leland Hill	D	22	14		1914	0.006		2.6	65	6.9	5	None	G			
				1	" "	T	285	6"		1917							None	D	10		*Collecting drain—85 ft. of 14" tile.
Tisbury	Municipal	1,822 W 3,000 S	0.268		Tashmoo Pond	S *	14	20x80		1887 1935			.04	7	5.8	11	None	C	800		

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Hardness	pH	CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Townsend	Municipal	500	0.132	36	Joslin-Weston Meadow	T	39	2½"		1934	0.432		.04	10	5.9	19	None	C	300		
Upton	Private	600	0.029	12	Adjacent to West River	T	41	2½"		1912	0.173		.07	39	6.1	24	None	C	200		
Uxbridge	Municipal	6,400	0.313	16	Old Group near West River	T	30	2½"		1904	0.260		.04	26	6.1	33	None	C	500		Discontinued 1940.
				17	New Group near West River	T	33	2½"		1923	0.250		.09	23	6.0						
Wakefield	Municipal	16,200	0.762	27	Saugus River Valley	T	30-50	2½"	S	1930-31	0.700		.24	61	6.2	—	None	D			Also Surface Supply.
Walpole	Municipal	8,400	1.344	10	Low Brook	T	28-70	2½"	G	1895)		)	.04	21	6.3		None	C	2,100		
				30	" "	T	28-70	2½"	G	1895)	1.240	)									
				101	" "	T	29-66	2½"	G	1918, 30	2.550	)									
Waltham	Municipal	43,600	3.560	1	Charles R. P.S. #1	D	30	41	G	1891			.83	45	6.5			D	3,475		
				1	Charles R. P.S. #2	D	35	30	G	1908			.03	33	6.4			D	3,475		
Ware	Municipal	4,700	0.384	1	Near Muddy	D	25	26	G	1887		)	.04	30	6.2	13.5	Kp	C	2,400		Can be flooded.
				41	Brook	T	35	2½"	G	1924		)									
Wareham	Fire District	6,500	0.285	25	Tihonet Pond	T	—	2½"	S, G	1907-35	0.500	)	.07	12	5.4	21		D	600		
				2	Tihonet Pond Outlet of	G	32, 37	18"	S, G	1940	0.540	)									
Warren	Water District	2,000	0.094	23	Comins Pond	T	45	2½"		1923			.07	15	6.2		None	D	400		
	W. Warren Water Co.	1,400	0.075	1	Adjacent to	D	18	20					.22	19	5.8		None	G			
				1	Tufts Brook			60		1915											
					" "	D	18	12		1920							None	G			
Wayland	Municipal	3,900	0.4190	30	Baldwin Pond	T	35-60	2½"	S, G	1925		)	.15	51	6.5		None	D	450		
				1	" "	G	48	6"	S, G	1933	0.500	)									
Webster	Municipal	13,650	0.705	1	Shore of Webster Lake	D	30	25		1893			.10	19	6.5	16	None	D	2,000		
				14		T	40	6"		1903							None	D			
				103	" "	T	40	2½"		1901) 1924) 1926) 1936)							None	D			
Wellesley	Municipal	17,600	1.752	33	Rosemary Brook	T	30-65	2½"	S, G	1898)	2.200		.04	44	6.5		)	D	1,390		
				17	" "	T	30-65	2½"	S, G	1915)							)				
				1	" "	T	31	10"	G	1924	0.800		.07	36	6.4			C	312		
				1	" "	D	40	25	S, G	1927	0.750		.04	38	6.5			D	1,110		
				1	Morse's Pond	G	69	24"			3.750							T			
									S, G	1937	2.140		.14	40	6.8						
West Brookfield	Municipal	1,387	0.091	10	Shore of Wickaboag Pond	T	27	2½"		1912	0.300	1912	.08	16	7.1		None	D	210		
West Stockbridge	Water Co.	425				S				1873			.05	38	7.1	5.0	None				

MASSACHUSETTS SURVEY DATA OF PUBLIC GROUND WATER SUPPLIES

City or Town	Name of Utility	Estimated Population Supplied	Present Consumption M.G.D.	Number of Wells	Location	Type of Well	Depth	Diameter	Well Ends in	Date of Construction	Test Yield M.G.D.	Date of Test	Analyses: Iron	Analyses: Hardness	Analyses: pH	Analyses: CO_2	Treatment	Type of Pump	Capacity of Pump	Present Daily Pumpage	Remarks
Westford	Abbott Worsted Co.	—		7	Gilson's Brook Valley	T	38-41	2½"	G	1920 1907)	.248		.04	31	6.5	22	None	D	87		
	Water Co.	2,300	0.113	19	Stony Brook	T	35-46	2½"	G	1917)	.500		.27	25	7.0	9	None	D	347		
				1	Meadow	D	34	?	G	1917)											
Westminster	Aqueduct Co.	225	0.010	1	Hillside S.W. of Village	S	9	12' 40'								5	None	G			
Weston	Municipal		0.290	1	Cherry Brook	D	22	10	S, G	1895		)	.10	56	6.4						
				11	(Warren Ave.)	T	22-25	2½"	S, G	1900		)									
				12	Stony Brook (Kendall Green)	T	25-28	2½"	G				.23	31	6.4						
				1	Riverside	G	70	24"	S, G	1941			.03	45	6.6						
	Water Co.			1	Charles R. (Well A)	D	25	12					.05	35	6.4			D			
				1	Charles R. (Well C)	D	30	20					.09	54	6.2			D			
Weymouth	Municipal	28,000 (3,300 W	1.565	1		G	65	24"			1.100		.05 (.03	35 135	7.6 7.6	1.0					Secondary Supply.
Williamstown	Water Co.	(4,270 S	0.399	2		S				1890			(.04	62	7.8		None	G			80% surface supply.
Wilmington	Municipal	7,500	0.223	42		T	45-85	2½"					.03	25	6.1	28	None	D			Well Field Flooded at Times.
Winchester	Municipal	15,300	1.124		Pond Street	T							.03	70	6.7						
Woburn	Municipal	19,750	1.504	1	Horn Pond	G	38	26"	G	1927			.13	52	6.7	13	None	T	1,040		
				3	Horn Pond	G	70	18"		1932)	1.000		.06	38	6.6	27	None	T	1,150	1.350	
						G	56	18"		1932)			.04	47	6.8	20	None	T	940	.900	
						G	64½	18"		1932)											
				1	Horn Pond	G	89	18"	G	1937	2.500		.03	54	7.0	15	None	T	1,870	1.700	
				1	Horn Pond	D	30	20	G	1908)	.500	)	.07	47	6.8	—	Dc	C	1,870		
				1	Horn Pond	G	17	12	G	1873)		)									
				1	Fowle Brook	G	40	24"	G	1937	1.300		.04	52	6.5	30	None	T	1,040	1.000	
Worthington	Fire Dist.	175 W 300 S		4	1 Mile N.W. of Town	S				1911			.18	11	6.4	2.0	None	G			
Wrentham	Municipal	5,100	0.228			T							.03	27	5.9						
				1	Franklin Street	G		24"					.05	14	5.9						
Yarmouth	Municipal	500 W 750 S	——	20	Union Street	T	35-47	2½"		1931, 33			.04	13	5.6	18		C	240		

City or Town	Owner	Population Supplied	Present Consumption MGD	Number of Wells	Location	Type of Well	Depth—Feet	Diameter
Burrillville Glendale New Village	Harrisville Combing Mills, Inc.	112		1	Isolated	T	419	
Burrillville Harrisville	Harrisville Fire District	5,000	.12	3	Isolated	A	42	6
				1	Near brook	A	131	8
Burrillville Mapleville	Stillwater Worsted Mills, Inc.	169-E 469	.015	1	Not Isolated	A	200	6
Burrillville Mohegan	Wanskuck Co.	60-E 260	.001	1 1	Near River	A	95	
Burrillville Oakland	Oakland Worsted Co.	180-E 360	.015	1 1	Near River Not Isolated	A A	160 230	6 8
Burrillville Oakland-Remington	Mrs. Earle Robinson	150	.015	2	Not Isolated	D	22	30
Burrillville Pascoag	Pascoag Fire District	3,500	.195	1	Isolated	G	46	12
Cumberland Lippitt Estates	Anderson & Barley	120-W 200-S	.006	5	Near Pond Isolated	T	30	2
East Greenwich	New England Water, Light & Power Ass'n	7,000	1.4	2	River Bank Isolated	G	118 107	12
Exeter	State of Rhode Island	1,000	.09	1 1	Isolated	D G	17 46	20
Hopkinton Alton	Richmond Lace Works, Inc.	135-E 435	.007	1	Not Isolated	D	12	120
New Shoreham Block Island	Est. of E. S. Payne	100-W 1,000-S	.1	3	Near Edge of Isolated Pond	T	37	3
North Kingstown	North Kingstown Water Commission	6,000-W 8,000-S	.230	1	Isolated	G	50	18
North Smithfield Forestdale	Forestdale Pump Co.	300	.030	1	Isolated	A	750	10
Smithfield Greenville-Greenlake	Northwestern Water Co.	200	.005	1	Isolated	G	50	6
Smithfield Greenville Terrace	Francis Foti	129	.0097	2	Not Isolated	A A	96 85	8 8
South Kingstown R. I. State College	State of Rhode Island	1,200-S 2,400-W	.259	1 1	Isolated	D A	26 85	6
South Kingstown Wakefield	New England Water, Light & Power Ass'n	6,000-S 4,000-W	.428	2	Isolated	G		
Tiverton North Tiverton	North Tiverton Fire District	2,500	.110	41	Isolated in Valley	T	24	2
Warwick & Coventry	New England Water, Light & Power Ass'n	19,000	1.5	19 4	Isolated	T G	32 56	2 16
Westerly Bradford	Bradford Dyeing Ass'n	1,200-E 1,600	.13	7	Near River, Isolated	T	63	2
Westerly Bradford	Town of Westerly	500	.027	25	Isolated low Area Near River	T	40 72	2
Westerly Westerly	Town of Westerly	19,500-S 15,000-W	.99	150	Isolated low Area Near River	T	30 72	2

Estimated Population Supplied
W—Winter
S—Summer
E—Employees

Type of Well
D—Dug
T—Driven
A—Deep well in rock
G—Gravel packed

Symbols Under Various
Well ends in
S—Sand
G—Gravel
R—Rock

NOTE: Does not include supplies serving fewer than 100 persons.

Diameter—Inches	Wells	Date of Construction	Yield Test GPM	Analyses				Treatment	Type of Pump	Capacity of Pump G.P.M.	Present Daily Pumpage M.G.	Remarks
				Iron	Hardness	pH	CO_2					
		1937		0.00	43	6.8	13					
6" 8"		1940	192	0.20	23	6.5	5	Dc	3-C 2-D	1,100	.12	
6"	R	1926	80	0.00	37	7.2	55	Dh	1-D	35	.015	
	R	1905	18	0.40	34	6.8	11		1-D	12	.014	
6" 8"	R	1911 1916	30	0.10	32	5.8	32		2-D	100	.01	
30"	G	1910 1930		0.05	45	5.5	65		2-D	21	.015	
12"	R	1942	250	0.05	33	5.9	35	Dc	1-D	250	.195	
2½"	G	1937		2.00	91	6.6	28	Dh	2-D	80	.006	
12"	G	1942 1943	3,440	0.10	23	7.2	15	Dc, Kp	2-D 1-C	2,000	2.0	
20"	G	1936 1947	32	0.00	18	5.6	40		1-D		.09	
120"	G	1916		0.00	34	5.6	38	Dh	1-D	20	.007	
3"	G	1935			31				2-C		1	Well supply is augmented by pond water in summer.
18"	G	1942		0.10	18	7.0	25	Dh, Kp	1-D		.18	
10"	R	1915		0.00	64	6.6	18		1-D	10	.03	
6"	G	1946	125	0.50	34	6.3	12	Dc	1-T	125	.005	
8" 8"	R R	1941 1937		0.05	34	6.0	32		3-D		.0097	
6"	G R	1935 1948	450	0.18	38	5.9	32		2-C		.259	
		1946		0.05	32	6.8	3	DhK	2-C		.55	
2½"	G	1928 1937	350	0.08	41	6.0	15	Dh	2-C 1-D	625	.11	Well field often flooded with surface water.
2½" 16"		1941 1945		0.10	25	5.7	13	Dh K	2-T 1-D	1,260	1.75	Well supply is augmented by pond.
2½"		1913		0.00	38	6.1	23		1-D	150	.13	
2½"	S	1937	875	0.00	25	5.9	24		2-C		.03	
2½"	G	1897 1943	680	0.00	26	6.1	23		1-T 2-D 1-C		.916	

rious Column Headings
in

Treatment
D —Disinfection
Dh—Disinfection by hypochlorite
Dc—Disinfection by chlorine
K —Chemical dosage for corrosive correction or water stabilization
Kp—Chemical dosage for corrosive correction or water stabilization by alkali feed for pH adjustment

Type of Pump
C—Centrifugal
D—Displacement
T—Turbine

Town	Utility	Population Supplied	Present Consump. MGD	No. of Wells	Location	Type of Well	Depth	Diameter
Alton	Alton Water Works	1,200	.072	1	In valley of river	G	40′	48″
Alton	Alton Water Works			1	On Hill	A	300′	6″
Bath	Bath Village Water Co.	145	.009	1	Marsh, Silty Soil	D	10′	10′
Belmont	Belmont Water Works	600	.082	1	Lowland near Valley	G	60′	48″
Bennington	Bennington Water Works	500	.038	3	On Hill	D	7′	50′
Campton	Beebe River Supply	550	.072	1	Foot of Hill	D	9′	55′x19′
Colebrook	Colebrook Water Co.	1,500	.099	10	River Bank	T	52′	2½″
Concord	Concord Water Works	25,000	3.0	150	River Valley	T	38.5′	2½″
Conway	N. Conway Precinct	2,000	.120	1	Brook Valley	D	18′	10′x10′
Derry	Derry Water Works	4,500	.225	10	Brook Valley	D	16′	(1-25″)(9-45″)
Dover	Dover Water Works	14,000	.975	1	Sandy Plain	G	82′	60″
				1	Near Lake	G	65′	48″
Epping	Epping Water Works	1,200	.079	16	River Valley	T	47-54′	2½″
Exeter	Exeter Water Works	5,398	.315	9		D	20′	(6-30″)(2-42″) (1-30″)
Farmington	Farmington Water Works	2,500	.097	1	Brook Valley	D	30′	20′
				1	Brook Valley	G	55′	48″
Francestown	Francestown Electric & Water Co.	150	.009	2	Hillside	D	12′	15′
Franconia	Franconia Water Works	450	.02	1	Side of Hill	D	10′	50′x100′
Franklin	Franklin Water Works	6,000	.24	17	Foot of Hill	D	6′	5′
Gorham	Gorham Alpine Aqueduct Co.	300	.018	6		D	10′x15′	10′
Greenville	Greenville Water Works	1,250	.044	1	River Valley	D	18′	20′
Hampton	Hampton Water Works Co.	3,500-W 15,000-S	.274	8		(5D)(3G)	23′	10′
Haverhill	Haverhill Precinct	400	.024	6	Sandy Plain	D	10′	8′
Haverhill	N. Haverhill Water Works	200	.03	7		D	10′	8′
Henniker	Henniker Water Works	1,200	.072	1	Foot of Hill	D	25′	25′
				1	Foot of Hill	T	40′	6″
Hill	Hill Water Works	400	.035	1	River Valley	G	45′	24″
Hudson	Hudson Water Company	2,500	.15	28	Brook Valley	26T & 2D	29′	D-2½″
Lyme	Lyme Aqueduct Co.	200	.012	1		D	8′	18′x20′
Marlboro	Marlboro Water Works	350	.017	7	Hillside	D	8′	10′x8′
Merrimack	Merrimack Company	245	.012	10	River Valley	T	18′	2½″
Merrimack	Shedd-Harris Mem. System	400	.02	1	Foot of Hill	D	10′	20′x20′
Milford	Milford Water Works			1	Brook Valley	G	45′	42″
Milford	Milford Water Works	3,600	.35	3	Brook Valley	D	12′	(1-18″)(2-30″)
Milton	Milton Fire District	600	.03	12	Near Lake	T	25′	2½″
Nashua	Pennichuck Water Works	30,000	2.63	21	Brook Valley	D	16′-52′	(19-2½″) (1-6″)(1-1½″)
Plymouth	Plymouth Water Works	2,200	.225	1	River Valley	D	16′	18′
Portsmouth	Portsmouth Water Works	26,000	1.51	50	Sandy Moraine	T	30′-50′	2½″
Portsmouth	Portsmouth Water Works			4	Sandy Moraine	T	60′	8″
Portsmouth	Portsmouth Water Works			1	Sandy Moraine	G	70′	36″
Portsmouth	Portsmouth Water Works			1	Sandy Moraine	G	70′	36″
Portsmouth	Portsmouth Water Works			1	Sandy Moraine	G	54′	36″
Portsmouth	Portsmouth Water Works			55	Sandy Moraine	T	35′-50′	2½″
Raymond	Raymond Water Works	725	.044	1	River Valley	D	30′	25′
Rollinsford	Salmon Falls Water Co.	800	.03	2	Brook Valley	D	202′-213′	8″
Somersworth	Somersworth Water Co.	6,200	.35	2	River Valley	G	70′	60″
Tamworth	S. O. Kimball Supply	100	.005	1	River Valley	D	7′	20′x12′
Wakefield	Sanbornville Water Works	550	.025	1	Brook Valley	D	18′	20′
Walpole	Walpole Water & Sewer Co.	500	.025	6	Brook Valley	T	20′	2½″
Walpole	N. Walpole Precinct	1,200	.060	3	Brook Valley	D	8′	8′
Whitefield	Whitefield Village Fire Dist.	1,200		4	Hillside	4A & 3D	(D8′-12′)	10″

	Well Ends in	Date of Construction	Yield Test G.P.M.	Analyses: Iron	Hardness	pH	CO_2	Treatment	Type of Pump	Daily Pumpage MGD	Capacity of Pump—G.P.M.	Remarks
	G	1938	700	0.05	44	6.5	30	—	T	——	200	Auxiliary Supply.
	R	1927	30		42				No Pump	——	No Pump	Free flowing well.
	S	1878		0.05	47	6.6	15	—	—	——	G	
	G	1938	500	0.05	20	6.3	25	—	T	.081	660	
	S			0.05	18	6.8		—	—	——	G	Augmented by Surface Supply.
3'	G	1926	500	0.05	18	6.8	6	—	D	.072	60	
"	S & G	1926		0.05	50	7.2	6	D	D	.098	250	Augments Surface Supply.
"	S	1930-31	1,500	0.10	10	6.4	8	K	C	——	1,000	Auxiliary Supply.
10'	S	1928			17	6.8	7	—	C	.108	100	Auxiliary Supply.
9-45')	S	1909-1940		0.15	60	6.3	37	D	C	.225	550	
	G	1940	970	1.0	37	6.6	13	D	T	.750	870	
	G	1947	350	.00	20	6.2	20	—	T	——	350	Not in service yet.
	G	1927		0.4	78	7.1	11	—	D	.079	147	
2-42")	S	1929-30		0.30	70	7.2	10	D	C	.315		Augmented by Surface Supply.
')												
	R	1938		0.05	20	6.3	25	—	C	.05	270	
	G	1943	300	0.05	20	6.2	25	—	T	.047	200	
	S	1922		0.05	25	6.6					G	
0'	S	1937		0.05	20	6.9		—			G	
	G	1938		0.05	42	6.3		—		.125	G	This source supplies 50% of use.
	S	1873			14	6.8	4	—			G	
	C	1915	40	0.02	20	6.6	19	D		.043	65	
	G	1908-1937		0.10	40	6.9	9	—	D & T	.1 to .9	1,580	
	S			0.00	45	7.4	4	—			G	
	C	1938	40	0.05	32	7.1		—		.03	G	
	G	1914		0.05	18	6.8		—	C	.036	165	
	S	1947		.00	20	6.7		—	C	.036	150	
	G	1940		0.15	27	6.7	7	—			150	
½"	S & G	1895, 1923, 1935		0.10	34	6.4	22	—	D		180	
20'	S			0.05	35	6.8	13	—			G	
8'	S	1908-1928		0.10	19	6.4	20	—		Gravity		
½"	S			1.20	25	6.6	18	A, F	D	.012	70	
20'	S	1934		0.05	50	7.3	3	—	C	.02	150	
'	R	1934	200	0.05	24	6.8	7	A, F		.35	550	
(2-30')	R	1889-1893	250	0.25		6.4	15					
½"	G	1936		0.05	20	6.8	10	K	C	.03	250	
½") -1½")	G	1898	4,500	0.10	31	6.9	2	D				Free flowing wells & springs.
	S	1918	210	0.05	27	6.2	24	K	C	.225	250	
½"	G	1892	200-350	0.70	73	7.4	6	D	D	2.5	1,740	
'	G	1889		0.05	59	7.6	3	D	D		700	
'	G	1922	500	0.05	73	7.7	3	D	T		420	
'	G	1922	450	0.05	81	7.7	3	D	T		125	
'	G	1934	220	0.60	47	7.4	4	D	T		240	
½"	G	1941		0.10	74	7.4	5	D	C		500	
	S	1930		0.05	37	6.3	23	—	C	.043	250	
'	R	1920	40	0.05	47	7.0	3	—	D		35	
'	R	1935-1939	600	0.10	31	7.0	3	K	T		600	
x12'	G	1937		0.05	18	6.4	20	—			G	
	S	1939		0.05	46	6.8	12	—	T	.025	150	
½"	S	1940		0.05	35	7.0	8	—	D		60	Auxiliary Supply.
	S			0.05	43	6.7	8	—	D		150	
	R	1935	325	0.00	26	7.2	3	—	T			

VERMONT—PUBLIC WATER SUPPLIES—WELLS

City or Village	Utility	Population	Consumption M.G.D.	Number of Wells	Location	Type	Depth	Ends in	Date	Analyses: Fe	Analyses: Hardness	Analyses: pH	Treatment	Pump	Cap. GPM	Remarks
Bennington Co. No. Bennington	Village	2,000	0.2	1	E. of S. Shaftsbury reservoir, in woods	D	14	G	1940		13	6.0	Kc	C	150	Part time operation.
Caledonia Co. Hardwick	Village	1,600	0.6	1	W. side village, in former sandpit	G	48	S	1940	.06	102	7.7	—	T	500	Auxiliary supply, can handle 100% of needs.
Lyndonville	Village	1,800	0.2	1	N. end village, E. side RR, flat area	G	54	S	1937		130	7.6	—	T	400	Auxiliary supply, can handle 100% of needs.
St. Johnsbury	Village	7,500	1.25	1	N. end village, E. side Passumpsic R., former gravel pit	G	unfinished		1949		250	7.6	unfinished			Auxiliary supply, mostly in summer.
Chittenden Co. Ft. Ethan Allen	U. S. Govt.	1,500	0.2	4	Flat plain around buildings and flat W. of Winooski R.	3A	400	3R			108	7.6	Dh	T		Normal supply.
						1G	30	1S	1941	.2	42	7.5	Dh	T	500	Auxiliary supply, was used 1941-45 when population sometimes reached 4,000.
Shelburne	Fire District	250	0.03	1	Center of village at school	A	330	R	1940	.28	140	7.6	—	T	60	100% of village supply.
So. Burlington	Fire District	250	0.03	1	Flat above Lake Champlain at Queen City	A	280	R	1946		167	7.7	—	T	100	100% of community supply.
Winooski	City	6,000	0.6	9	Around reservoir, N.E. side city	3A	450	R	1905		106	7.5	Dc	T		Normal supply in combination with surface supply.
						6T	15	S	1949		69	7.6	Dc	C	200	New auxiliary well field.
Lamoille Co. Morrisville	Village	2,000	0.15	1	N. of village on S. side Lamoille R.	G	30	S	1942		70	7.0	—	T	500	About 80 to 90% of normal supply in combination with a few small springs
Orange Co. Randolph	Village	2,000	0.2	1	Center of village, S. of RR.	A	196	R	1941		113	7.4	—	T	400	Auxiliary supply, used mostly in summer.

VERMONT—PUBLIC WATER SUPPLIES—WELLS

City or Village	Utility	Population	Consumption MGD	Number of Wells	Location	Type	Depth	Ends in	Date	Analyses: Fe	Analyses: Hardness	Analyses: pH	Treatment	Pump	Cap. GPM	Remarks
Wells River	Village	550	0.06	1	In village	A	84	R	1897	.1	119	7.7	—			Partial supply, used 100% at times.
Orleans Co. Newport	City	4,500	0.9	1	N.E. part of city, on E. side South Bay	G	47	G	1949		75	7.7	—	T	300	Auxiliary supply, use started late in 1948.
Rutland Co. Castleton	Village	400	0.06	1	N. side village, flat S. of Castleton R.	G	31	G	1939		75	7.0	—	T	400	100% of village supply.
West Rutland	Village	2,500	0.3	1	E. side village, flat area	T	20	G			136	7.7	—	C	100	Auxiliary supply, used in late summer, if needed.
Washington Co. Northfield	Village	2,200	0.3	2	S. of village, flat on E. side Dog R.	G	30	S&G	1939 & 1948	.02	45	6.8	Dh	T	400 & 400	1 well used continuously as partial supply, 1 well for auxiliary use.
Waterbury	Village	3,300	0.3	1	Center of village, in park	G	32	G	1938	.1	105	6.5	Dc	T	500	Auxiliary supply.
Windsor Co. Springfield	Village	6,000	0.7	30	N. of village, flat on W. side Black R.	T	25	S	1941		65	7.3	—	C	750	90% of supply in summer, auxiliary at times.
Windsor	Village	3,500	0.5	2	N.W. side of village, N. of pumping station	G	25	G	1947 & 1948		120	7.7	—	T	400 & 400	1 well used majorit of time as partial supply, 1 well auxiliary.

PROCEEDINGS

FEBRUARY 1949 MEETING

HOTEL STATLER, BOSTON, MASS.

Thursday, February 17, 1949

President Arthur C. King in the Chair.

Secretary Knox announced the election of the following new members: Nathaniel Clapp, Project Engineer, Metcalf & Eddy, Boston, Mass.; Arthur L. Dopmeyer, Sanitary Engineer Director, U. S. Public Health Service, Boston, Mass.; Martin J. Markham, Engineer, Department of Public Works, Stoneham, Mass.; Minot P. Second, Superintendent, Water Department, Winchester, N. H.; Edwin J. Titsworth, Koppers Company, Inc., Boston, Mass.; Leonard H. White, Professional Engineer, Worcester, Mass.; Mathew D. Sullivan, Assistant Superintendent, Water Department, Fall River, Mass.

A paper on "The New Year's Flood of 1949 in Western Massachusetts" was read by Angelo Iantosca, Junior Sanitary Engineer, Massachusetts Department of Public Health, Boston, Mass.

A paper on "Effects of Municipal Water Treatment on Boiler-Water Conditioning" was read by J. N. Welsh, Director of Engineering Service, Hall Laboratories, Inc., Pittsburgh, Pa.

MARCH 1949 MEETING

HOTEL STATLER, BOSTON, MASS.

Thursday, March 17, 1949

President Arthur C. King in the Chair.

PRESIDENT KING. I am sorry to announce the death during the past month of one of our members, and I shall ask you please to rise for a moment, in respect to the memory of Ernest J. Sullivan. [*The members stand.*]

Ernest J. Sullivan, Assistant Sanitary Engineer of the Massa-

chusetts Department of Public Health, died on February 21 at the age of 52. He had been an active and enthusiastic member of the New England Water Works Association since 1934, and was very well esteemed by all who knew him. Ernest Sullivan was a native of Maine, graduated from the University of Maine, and also attended the Harvard Graduate School of Engineering. His engineering career began with Weston & Sampson, followed by 27 years of service with the Massachusetts Department of Public Health.

The Secretary has some announcements to make.

SECRETARY KNOX. At the Executive Committee meeting this morning, nine new members were admitted to membership, as follows:

Members: Charles A. Duffy, Inspector, Water Department, Boston, Mass.; Chester H. Minehan, Assistant Hydraulic Engineer, U. S. Geological Survey, Boston, Mass.; Richard J. Kelley, Inspector, Water Department, Boston, Mass.; Albert P. McCann, District Foreman, Water Department, Boston, Mass.; Nathan S. Bubbis, General Manager, Greater Winnipeg Water and Sanitary Districts, Winnipeg, Manitoba, Canada; Wilbur H. Dewey, Superintendent, Biddeford & Saco Water Co., Biddeford, Me.; Ernest H. Ebert, Superintendent, Water Department, North Attleborough, Mass.; Charles Y. Hitchcock, Jr., Project Engineer, Metcalf & Eddy, Boston, Mass.

Associate Member: General Painting and Maintenance Corp., Boston, Mass.

PRESIDENT KING: According to the Constitution and By-Laws of the Association, at the March meeting we have to elect a Nominating Committee. That committee is charged with the duty of submitting a list of candidates for officers for the ensuing year. I await your pleasure.

RICHARD H. ELLIS. I move that the Nominating Committee consist of the five latest Past-Presidents, excluding those on the Executive Committee, and that they be Horace L. Clark, Chairman, Leland G. Carlton, Francis H. Kingsbury, Karl R. Kennison and Harold W. Griswold.

STEPHEN H. TAYLOR. I second the motion.

(Upon motion duly made and seconded, it was voted that the nominations be closed. The question was then put and the motion carried, and the Past-Presidents named were declared elected as a Nominating Committee.)

PRESIDENT KING. Upon the suggestion of Walter C. Conroy that the Association provide an award in addition to the Dexter Brackett Memorial Award for a meritorious paper, the matter has been considered by a committee, which has reported to the Executive Committee. The purpose of such an award—which is to be known as the Commemorative Award—is to give due recognition to the paper which is adjudged as the next most meritorious to the Dexter Brackett Memorial Award, and it will perpetuate the memory of members prominent in the water-works profession.

In this connection, suggested rules governing the Commemorative Award have been drafted for your consideration, and I shall now ask the Secretary to read those rules, prior to taking a vote as to the establishment of the Commemorative Award and the adoption of the suggested rules governing the award.

SECRETARY KNOX reads the suggested rules.

PRESIDENT KING. The first question is on the establishment of this Commemorative Award. Is there any discussion?

(Upon motion duly made and seconded, it was VOTED that a Commemorative Award be established by this Association.)

PRESIDENT KING. The next question before the Association is the establishment of the suggested rules governing this award, as just read by the Secretary. Is there any discussion on that?

LINN H. ENSLOW. Gentlemen, I am probably very much out of order in discussing this matter before you, but I would like to suggest to the Committee that, in addition to the award of these fine books, they add a certificate which the recipient can frame and hang in his office.

THOMAS R. CAMP. I would like to move that the rules be adopted with the amendment to include a certificate.

PERCY A. SHAW. I second the motion.

PRESIDENT KING. You have heard the motion, that the rules be adopted with an amendment to include a certificate as well as the books.

(The question was put and the following amended rules were adopted.)

RULES GOVERNING THE COMMEMORATIVE AWARD

Committee on Awards

1. The Committee on Awards for the Dexter Brackett Memorial Medal shall also recommend the recipient of the Commemorative Award for the previous year.

2. Those papers considered by the Committee on Awards which shall remain after the selection of the Dexter Brackett Memorial Medal paper shall be eligible for the Commemorative Award.

3. The Committee shall report its recommendations to the Executive Committee on or before August 1, and the award shall be made by the Executive Committee. The announcement of the award and the presentation thereof shall be made at the Annual Convention.

Rules for Award

1. Competition for the award shall be restricted to the members of the New England Water Works Association.

2. The award shall consist of a book or books, of a value not exceeding fifteen dollars, to be selected by the Committee on Awards, who may consult with the recipient as to his desires in the matter. The book or books shall be suitably inscribed on the flyleaf with the name of the recipient, the title and date of presentation of the paper in consideration of which the award is made, together with the name of the member or members of the Association whose interest in the particular field of the profession treated in the paper is commemorated by the award. A suitably inscribed certificate of the award shall be presented to the recipient.

3. The Commemorative Award shall be made for the most meritorious of the papers eligible therefor. Due consideration shall be given to the applicability of the content to general water-works problems and practices.

PRESIDENT KING. We now come to a very pleasant duty and pleasure, which is not of frequent occurrence, and that is the presentation of honorary membership to one of our valued members.

E. Sherman Chase has been a member for quite some years. He has always been very active and prominent in our work, and I am sure that all the time his thought and work has been with the good

of this Association in mind. He joined this Association on September 7, 1920. He was President in 1933-1934.

Mr. Chase was educated in the Merrimack public schools and the Massachusetts Institute of Technology. He has been actively engaged in his profession of sanitary engineering since 1907. His first association was with Professor L. P. Kinnicutt at Worcester Polytechnic Institute in 1907-1908; then with Hering & Fuller. From 1913 to 1920 he was Assistant Sanitary Engineer with the New York State Department of Health. From 1920 to 1927 he was Sanitary Engineer with Metcalf & Eddy and since that time has been a partner of that firm.

Mr. Chase has served on many program, war-time and other committees of this Association. Probably his most noteworthy contribution to this Association and to the water-works fraternity has been his sponsorship and successful development of the plan for co-operation and interchange with the Institution of Water Engineers in Great Britain. He has recently been elected to honorary membership in the Institution.

I think it is particularly fitting that we should present him with honorary membership in this Association at this time, inasmuch as in the near future he is to visit over there, and I am sure we shall be glad to have him go as an honorary member of this Association.

Mr. Chase, it is a very great pleasure to me to be able to present you with this certificate and to say that we are very happy to have you as an honorary member of this Association. [*Applause.*]

E. SHERMAN CHASE. Mr. President and fellow members: It would be foolish for me not to admit frankly that I am most pleased and gratified with this action on the part of the Association. I think without exception it is one of the pleasantest experiences of my life.

The New England Water Works Association has had a very long and very distinguished career, and that alone is sufficient to make one very happy indeed to be made an honorary member of it. But, more than that, the New England Water Works Association stands for friendship, and it is the friendships that we have made and are making in this Association that are to me the most important part of membership in this Association.

From the bottom of my heart I thank you most sincerely. [*Applause.*]

PERCY R. SHAW. I would like to move that Mr. E. Sherman Chase be commissioned by the New England Water Works Association to take the cordial greetings of this Association to the Institution of Water Engineers at their meeting in England this summer.

[The motion was duly seconded.]

PRESIDENT KING. The motion is that Mr. Chase be instructed to take the greetings of this Association to the Institution of Water Engineers, when he visits England. Is there any discussion?

[The question was put and the motion was carried.]

I am sure that Mr. Chase will be glad to do that.

A paper on "Algae Control at Danbury, Connecticut," was read by Ellis A. Tarlton, Chemist, Water Department, Danbury, Conn.

A paper on "Some Problems in Water-Works Construction" was read by Ralph H. White, R. H. White Construction Co., Auburn, Mass.

A paper on "Ground-Water Supplies in Boston and Vicinity" was read by Henry N. Halberg, Engineer in Charge, U. S. Geological Survey, Boston, Mass.

ROSTER OF SANITARY AND PUBLIC HEALTH ENGINEERS*

The following is of importance to all sanitary and public health engineers:

Efficient utilization of our national resources, both human and otherwise, is difficult during periods of national emergency. Criticism has been directed against the federal government and its agencies for the inefficient use of sanitary and public health engineers during World War II.

The National Security Resources Board is charged with the responsibility of planning for the efficient use of our national resources in any emergency. James A. Crabtree, Medical Director, U. S. Public Health Service, has been assigned to the NSRB to develop methods for the best utilization of medical and public health

*Reprinted from *American Journal of Public Health*, Vol. 39, No. 3, March, 1949.

personnel and facilities. One of the first goals of Dr. Crabtree's Medical Division is the preparation of comprehensive rosters of professional personnel. Should another national emergency arise, these rosters will be used in developing programs that will assure the best use of all professional people.

Arrangements have been made with the American Public Health Association, through the Engineering Section Project, for the preparation of a roster of all sanitary and public health engineers in the United States, other than those in federal service. The magnitude of this task can be appreciated when it is realized that there is no comprehensive list available similar to the membership list of the American Medical Association, the American Nursing Association, or the American Dental Association. Neither are there state professional registers such as exist for physicians, nurses, and dentists.

Lists of engineers are now being prepared from membership lists of the various professional and technical societies. The Conference of State Sanitary Engineers has volunteered its active support and assistance. Other organizations and even individuals are being asked to help. Questionnaires will be sent out to individuals in the near future. Prompt return of the forms is requested. It will be to the advantage of each engineer to assure his proper classification so that proper use of his training and experience can be obtained. Any engineer not receiving a questionnaire by July 1 should communicate directly with the

Engineering Section Project
American Public Health Association
1790 Broadway
New York 19, N. Y.

so that his name can be entered on the roster and he be furnished with a copy of the questionnaire.

Undoubtedly there are individuals professionally qualified in sanitary or public health engineering who are not now actively engaged in that field. Since they may not belong to any of the more important professional or technical engineering societies they may not see this notice. You are urged to advise the Engineering Section Project of the names and addresses of such people.

INDEX OF ADVERTISERS

	Page
Atlas Mineral Products Co.	xxvi
Badger Meter Mfg. Co.	Following front cover
Bailey, Howard E.	ii
Barrett Division, The (Allied Chemical & Dye Corp.)	Following front cover
Barrows, H. K.	ii
Boston Edison Co.	ix
Builders-Providence, Inc. Division of Builders Iron Foundry	xii
Camp, Thomas R.	ii
Cast Iron Pipe Research Assn.	xxxi
Cement Lined Pipe Co.	iii
Centriline Corporation	Following front cover
Eddy Valve Co.	xix
Edson Corp., The	xxi
Eureka Pipe Co., Inc.	xxvi
Fairbanks-Morse Co.	xi
Fay, Spofford & Thorndike	ii
Flexible Sewer Rod Equipment Co.	xxii
Ford Meter Box Co.	xvi
Haley & Ward	ii
Hays Mfg. Co.	xvii
Hersey Mfg. Co.	iv
Hughes, Edward F., Co.	iii
Hydraulic Development Corp.	x
Johns-Manville	Following front cover
Kennedy Valve Mfg. Co.	xiii
Knowles, Morris, Inc.	ii
Koppers Company, Inc.	Following front cover
Layne-New York Co., Inc.	iii
Leadite Co.	Back cover
Leggette, R. M.	ii
Lock Joint Pipe Co.	Facing front cover
Ludlow Valve Mfg. Co.	xxviii
Maher, D. L., Co.	iii
Metcalf & Eddy	ii
Morris Machine Works	xviii
Mueller Co.	Following front cover
National Lead Co. of Mass.	xv
National Water Main Cleaning Co.	viii
Neptune Meter Co.	vii
Omega Machine Co. (Division of Builders Iron Foundry)	v
Pierce-Perry Co.	iii
Pipe Founders Sales Corp.	xxii
Pirnie, Malcolm	iii
Pitometer Co.	iii
Pittsburgh Equitable Meter Co.	Following front cover
Potts, Clyde	ii
% Proportioneers, Inc. %	Following front cover
Red Hed Mfg. Co.	xxiv
Rensselaer Valve Co.	xxvii
Reppucci, C. & Sons, Inc.	iii
Ross Valve Mfg. Co.	xxiii
Smith Mfg. Co., The A. P.	Following front cover
Turbine Equipment Co.	xiv
U. S. Pipe and Foundry Co.	xxxii
Utilities Supply	xxii
Wallace & Tiernan	xx
Warren Foundry and Pipe Corp.	xxi
Welsbach Corporation	xxv
Weston & Sampson	iii
Whitman & Howard	iii
Wood, R. D., Co.	Following front cover
Worthington Pump and Machinery Corp.	vi

(Classified index on page xxix)

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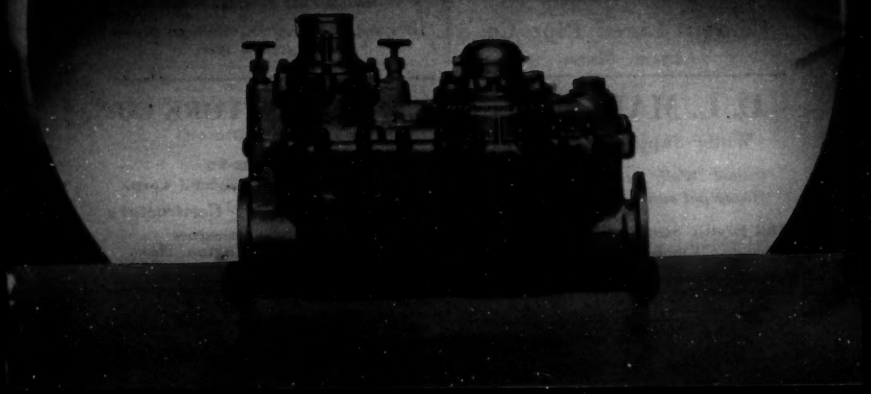

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Rotolock Feeder	Bulletin 50-53
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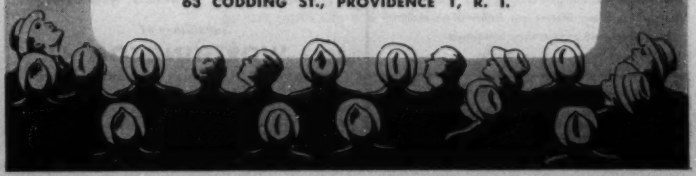

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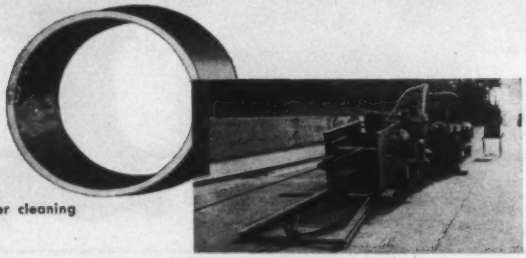

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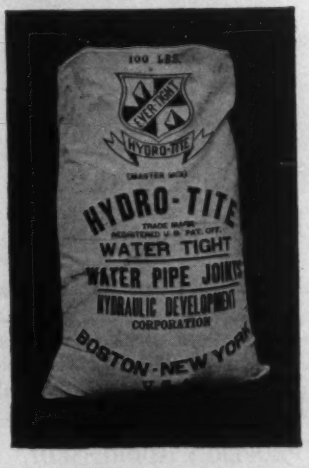

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For more than thirty years water works superintendents have been using Hydro-tite for joining bell and spigot pipe. They have found that this self-caulking, self-sealing compound costs less to buy, requires less labor, makes tighter joints and withstands the effects of time, strain and vibration. Send for data book and sample.

Always use Fibrex, the bacteria-free packing for pipe joints. Send for sample.

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Works: West Medford Station, Boston, Mass.

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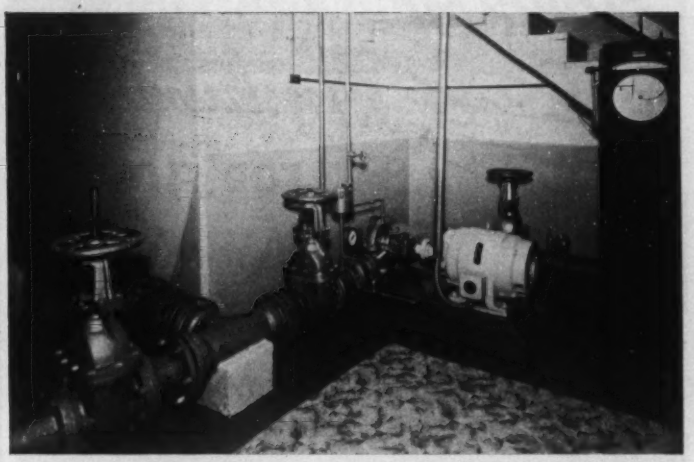

One of the two Fairbanks-Morse motor driven Centrifugal pumping units in HUDSON, Mass., Municipal Pumping Station.

Town officials at Hudson, Mass., are pleased with the "Undivided Responsibility" they have obtained through the team-work of their Fairbanks-Morse Pumps, driven by Fairbanks-Morse Motors.

When additional gallonage became necessary to supply the increased population and care for industrial expansion, the Town of Hudson drove a well field and installed two Fairbanks-Morse Centrifugal Pumps, driven by Fairbanks-Morse Motors. Installed by the R. P. Hall Company of Boston, these Pumps are now giving Hudson low-cost Pumping efficiency.

A trained Fairbanks-Morse Engineer is on call to help solve YOUR Pumping problems.

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QUESTION { What hydrant can be repaired by one man in only 30 minutes after collision damage?

ANSWER { THE **KENNEDY SAFETOP** HYDRANT

Inexpensive, quickly replaceable standpipe breaking ring and stem coupling take the blow . . . and the damage!

One man with socket wrench, large screw driver and light sledge hammer makes repairs in 30 minutes.

Complete interchangeability of all parts of each size lowers maintenance expense.

No flooding when broken . . . protects property. No need to shut off water during repair.

All adjustable parts enclosed within cap to prevent tampering or theft.

Double automatic drain valve assures complete drainage, prevents freezing; automatically closes as hydrant opens.

Easily extended for raise in street grade . . . without excavation or service interruption.

Scientific design of entrance elbow, without projections or pockets, decreases pressure drop.

Conforms to the specifications of the American Water Works Association.

Straight line operating mechanism minimizes pressure loss from inlet elbow to nozzles.

Extra large heavy standpipe cuts friction loss . . . withstands ordinary impacts.

Write for Bulletin 62

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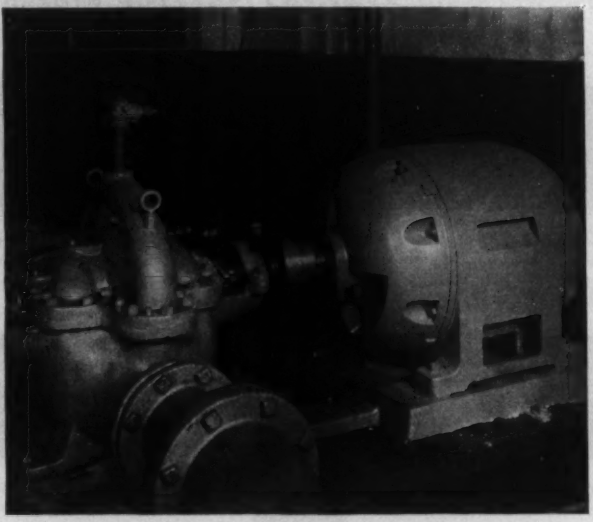

Town of Clinton, Mass., Water Works

A recent installation of a modern heavy duty De Laval water works pump designed for 1500 G.P.M., against 232 feet net head with 65 feet positive head on the suction. Special sleeve construction is employed because of the high positive suction pressure.

This new unit replaces an old De Laval pump that was retired after more than 25 years of satisfactory service because of increased water demand.

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New England Representatives for

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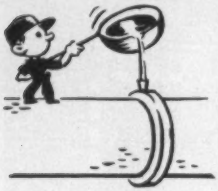

Ingot or Wool?

Whether the caulking job calls for ingots for cast lead joints or lead wool for cold caulking, the primary requirement in either case is lead of high purity.

Don't take chances with inferior caulking lead which may have been manufactured from scrap material containing a large percentage of impurities.

When present in a caulked joint, impurities on the surface will corrode and cause weak spots. The stresses due to normal pipe movement concentrate at these areas and failure results.

Our lead, both in ingot and wool form, is made from selected metals and carries the Lead Industries Association Seal of Approval as a guarantee of its quality.

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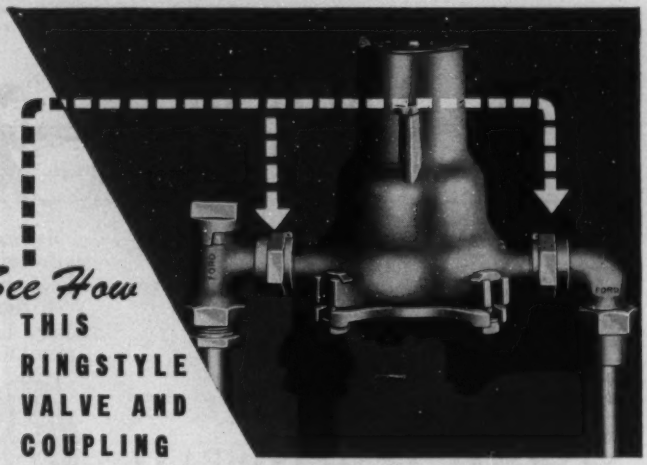

See How THIS RINGSTYLE VALVE AND COUPLING

CUT Installation Time!

Any water works man can see in a flash how Ford Ringstyle valves and couplings save pipe-fittings and installation time.

The meter nut is attached to the valve and the coupling by Ford's ingenious Ringstyle construction .... a sturdy bronze pin is driven through the nut and forms a ring behind the flange.

These fittings are described in our new catalog No. 50—it is FREE.

THE FORD METER BOX COMPANY, INC.

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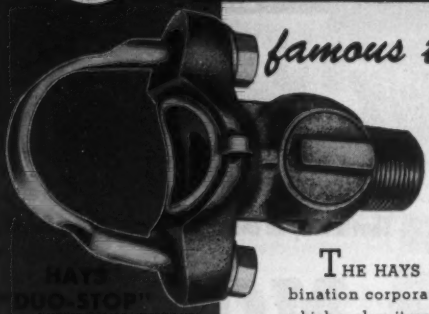
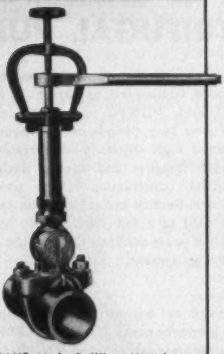
WATER SERVICE PRODUCTS

famous throughout the world



THE HAYS "Duo-Stop" is a combination corporation stop and saddle which makes it unnecessary to buy corporation stops and service clamps or saddles separately. It is designed to install on 2 inch mains, either cast iron or steel . . . requires no tapping of the main and can be installed without interruption of service. It can be supplied with outlet for iron pipe, copper service, or various types of thread.

Hays Manufacturing Company is one of the largest making a complete line of water service products, and throughout eighty years the name HAYS has been a guarantee of quality. Every HAYS fitting is precision machined, specially lubricated for enduring operation, and hydrostatically tested at 200 pounds or more. All corporation stops can be installed with any standard tapping machine equipment.



HAYS Main Drilling Machine for installing HAYS "Duo-Stop." Ask for our new complete catalog of water service products.

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✓LOW POWER COSTS
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Sustained Quietness and Efficiency...

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Wilmington, N. C.

Rugged in construction . . . with design stresses low for smooth, trouble-free operation . . . MORRIS CENTRIFUGAL PUMPS run quietly and uninterruptedly over long periods of continuous operation. Sustained high efficiency—the result of correctly related impeller and casing designs plus Morris' quality construction—keeps power consumption down. Service and repair costs per million gallons are held to a few cents. Daily experience over a period of years confirms the wisdom of selecting MORRIS equipment.

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Consult our engineers now regarding your particular needs. There's a MORRIS pump for every job, and more than three-quarters of a century of pump-building "know how" behind every MORRIS pump.

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EDDY

Valves and Hydrants

Distinctive Features of the EDDY Valves

ONLY three parts are moved by the Stem—the ball and the two gates. Each gate is hung from one of the trunnions on the ball. A convex surface at the base of each trunnion fits a concave surface on the back of the gate. This allows the gates to adjust themselves properly to their seats. . . . The gates, being free to revolve on the trunnions, do not always seat in the same position. Gates are center bearing and adjustable. They are forced to their seats with equal pressure at all points. There are two hooks on the ball on sizes 4-inch and larger, which loosely engage with the gates.

Distinctive Features of Our New Swivel Top Hydrants

THIS new hydrant has all the advantages of the popular EDDY fire hydrant, plus several additional features. . . . Nozzles are in a short flanged section of the standpipe, which may be unbolted and turned to different positions. . . . Should the standpipe be broken, only the casting below the swivel head need be replaced.

. . . To raise the hydrant to conform to a new grading, simply insert a flanged extension piece below the swivelhead. No digging. To add a steamer nozzle at any time, it is only necessary to replace the swivel head with one having a steamer nozzle.

Besides valves and hydrants to meet the most exacting demands of waterworks service, the complete EDDY Line includes: valves for sewage disposal works; valves for steam, gas and oil; check valves, foot valves, plug valves, shear gates, indicator posts, etc. EDDY valves are made in three classes: Iron Body Bronze Mounted, All Bronze, and All Iron Valves.

Eddy Valve Co.

WATERFORD NEW YORK

Two Birds with One Stone...

AT LITTLE ROCK

In the Little Rock Water System, organic growths reduced the capacity of a large reinforced concrete supply line by twenty percent.

According to L. A. Jackson, Manager, and W. A. Mayhan, Chemist, in a joint report appearing in Water & Sewage Works, the Break-Point Process when applied to this line greatly reduced organic growths and gave better results than any other method tried in a three-year test period.

Reduction of organic growths was not the only benefit however. According to the report "•••Aside from destroying the organic growth in the transmission line, 'break-point' chlorination has proved of value to us in other ways. Since beginning this treatment, taste and odor flare ups have been practically nonexistent. We are assured a sterile water throughout the entire plant and lactose-fermenting organisms in the plant are a thing of the past.•••"

This experience at Little Rock is representative of the multiple protection that you can obtain with the Break-Point Process. For complete details on other Break-Point benefits such as longer filter runs and iron and manganese removal, see your nearest W&T Representative.

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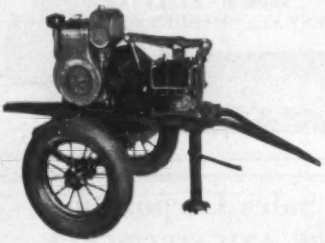

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1879—ROSS— 1879

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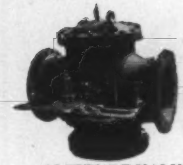


Controls elevation of water in tanks, basins and reservoirs

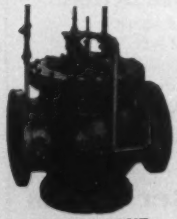
ALTITUDE VALVE

1. Single Acting
2. Double Acting

Maintains desired discharge pressure regardless of change in rate of flow

REDUCING VALVE

Regulates pressures in gravity and pump systems; between reservoirs and zones of different pressures, etc.



Maintains levels in tank, reservoir or basin

1. As direct acting

2. Pilot operated and with float traveling between two stops, for upper and lower limit of water elevation.

FLOAT VALVE

A self contained unit, with controls three or more automatic

COMBINATION VALVE

Combination automatic control both directions through the valve.

Maintains safe operating pressures for conduits, distribution and pump discharge

SURGE-RELIEF VALVE

Electric remote control—solenoid or motor can be furnished

REMOTE CONTROL VALVE

Adapted for use as primary or secondary control on any of the hydraulically controlled or operated valves.

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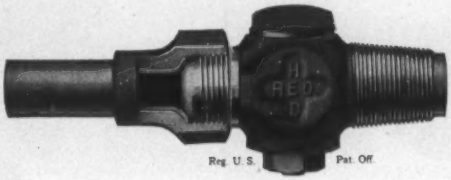


RED HED Curb Stops and Corporations are made with *hand ground* plugs lapped to the bodies. This method of manufacture insures consistently easy turning, freedom from failure and costly replacements.

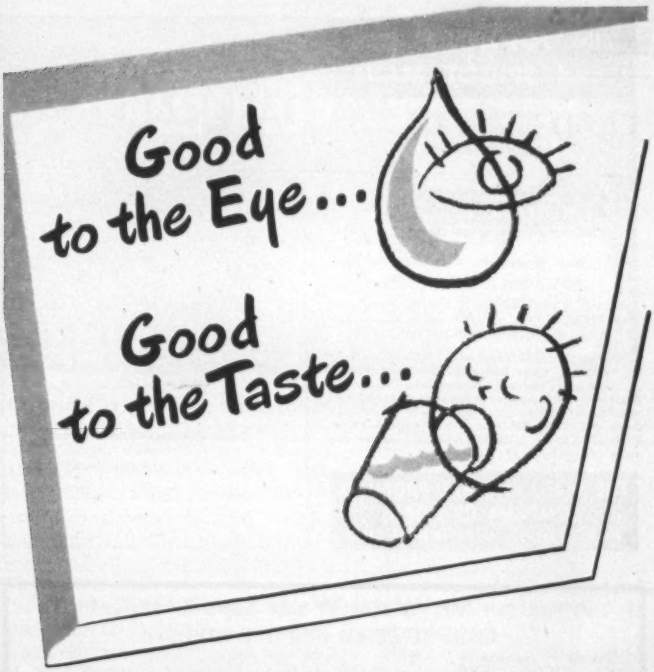
Many users of RED HED Brass Goods can verify these statements. RED HED Brass Goods are less costly because they are better.

RED HED MFG. CO.

Manufacturers of

RED HED Brass Goods for Water Works

368 Congress Street **Boston 10, Mass.**



Ozonated water is both good to look at and good to taste. Communities where unpleasant taste, color and odor removal problems persist in drinking water are giving increasing attention to Welsbach Ozonators as the answer.

Welsbach—a name long noted for sound engineering—has been identified with America's largest Ozone installations for the purification of municipal water supply. The recent record-breaking installation at Philadelphia forms the basis for an informative and interesting booklet whose reading may be of assistance to you. It will be sent you gladly on your request—without obligation to you, of course. Write for "Philadelphia Finds the Answer" today.

Address: Municipal Water Systems Research Dept.

Ozone Processes

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OZONE PROCESSES DIVISION

1500 Walnut Street **Philadelphia 2, Pa.**

IN JOINTING BELL and SPIGOT MAIN

LEAD costs around 25¢ a lb. . . . . and, for a 2" joint on 8" pipe, you'll need about 13¾ lbs.

Tegul-MINERALEAD

costs around 9¢ a lb. and, for that same joint, you'll need only 4 lbs.

Figure it out and you'll find you can save approximately 89% of your material costs by switching to Tegul-MINERALEAD.

Add to this that Tegul-MINERALEAD needs no caulking, deep bell holes or skilled labor . . . and gives lastingly tight joints with ample flexibility and super-resistance to thermal and mechanical shock . . . and you'll agree that Tegul-MINERALEAD is a SUPERIOR COMPOUND FROM EVERY ANGLE. Write for more information.

THE ATLAS MINERAL PRODUCTS COMPANY OF PENNA
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WORCESTER, MASSACHUSETTS

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CEMENT LINED SERVICE PIPE:—

Resists Corrosion

Has proven practical for nearly three-quarters of a century

Immediate delivery can be made to waterworks on both wrought iron and steel, black or galvanized.

"SPECIFY EUREKA PIPE"

EUREKA PIPE COMPANY, Inc.

Manufacturers of CENTRIFUGAL CEMENT LINED PIPE and LEAD AND TIN LINED FITTINGS

591-593 WASHINGTON ST. TEL. 3-9550 LYNN, MASS.

WHY FAMOUS

RENSSELAER

DOUBLE DISC

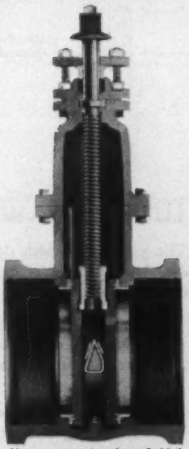
GATE VALVES

open easier... seal tighter!

IN OPENING, THE THICK (A) SIDE OF THE STEM NUT LIFTS THE GATE ON ONE SIDE FIRST, MECHANICALLY BREAKING SEAL

GATE ON SHORT SIDE (B) OF WEDGE IS PULLED FIRST, LOOSENS EASILY. IN CLOSING, LONG SIDE (C) OF WEDGE CAUSES POWERFUL SEATING FORCE ON THAT SIDE.

The principles of the long Wedge for greater seating power and the short Wedge for easier unseating apply to RenSSelaer Valves in sizes 10" and larger. But in ALL sizes of these famous Valves, ONE gate is ALWAYS lifted FIRST, mechanically releasing the wedges, so opening is positive. Wedging action does not seat gates until they are in *"fully closed"* position, so sliding frictional wear is minimized.



Above cross-section shows 2-sided wedge on valves 10" and larger

But ease of operation is only one feature of RenSSelaer Valves. SOLID BRONZE is used liberally throughout – in stems, stem nuts, wedges, seat and gate rings, glands and stuffing box nuts. Castings are high tensile strength corrosion-resistant iron. INTERCHANGEABILITY of parts insures low maintenance cost. Thousands in perfect service for over 65 years.

Experienced Field Representatives are at your call to bring you the benefit of RenSSelaer experience in Valves AND Hydrants. Their service puts you under no obligation. Call our nearest office.

LEADERSHIP FOR OVER 65 YEARS

RENSSELAER

VALVES AND HYDRANTS IN STOCK
ATLANTA
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KANSAS CITY
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Gate Valves • Square Bottom Valves • Tapping Valves and Sleeves
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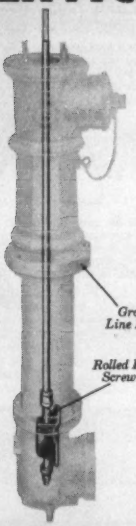
SERVICE . . . with a monkey wrench!



To inspect or service a Ludlow Slide Gate Hydrant you'll need a wrench — that's all. Just take off the top of the hydrant and lift out the stem, *without unscrewing anything below ground.* The entire mechanism, including the gate, bronze wedge nut and drip assembly is attached to the stem.

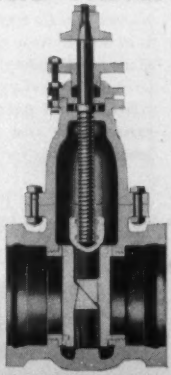
LUDLOW FIRE HYDRANTS

Shown here is the distinctive Ludlow List No. 90, now serving in thousands of cities and towns. It employs the famous Ludlow Slide Gate principle, and (in addition to easy servicing) offers the following advantages:

- *Quick Water* with least possible shock and pressure loss. Proper shut-off without water hammer.
- *Proper Drainage*, automatically, at lowest point when closed.
- *No Flooding.* Wedge-locked when closed. Accidental breaking will not cause flood. Hydrant head breaks at ground line — is easily replaced without excavation.

LUDLOW AWWA DOUBLE DISC GATE VALVES

The parallel seated, double wedge type slide gate valve, developed and perfected by Ludlow, provides a number of essential benefits.



- *Smooth, Positive Operation.* Gates positioned directly opposite ports before wedging, and entirely unwedged before being raised.
- *Positive Closure.* Even after years of service in the open position; flexible-action gates self-adjusting to seats.
- *Simplicity!* Only 2 parts in the wedging mechanism! These are extra heavy with ample reserve strength and wear resistance for years of constant service.

Made in sizes from 2″ to 72″, N.R.S. and O.S.&Y. with all types of connections. 95-S

TRADE LUDLOW MARK

The LUDLOW VALVE MFG·CO·INC·TROY·N·Y·

SINCE 1866

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AIR COMPRESSORS.
Edson Corp., The xxi

ASBESTOS CEMENT PIPE.
Johns-Manville Corp. ..... Following front cover

BRASS GOODS.
Eureka Cement Lined Pipe Co. xxvi
Hays Mfg. Co. xvii
Mueller Co. ..... Following front cover
Pierce-Perry Co. iii
Red Hed Mfg. Co. xxiv
Smith Mfg. Co., The A. P. ..... Following front cover

CALKING MACHINERY AND TOOLS.
Hydraulic Development Corp. x
Mueller Co. ..... Following front cover

CAST IRON PIPE. (See Pipe, Cast Iron.)

CEMENT LINED PIPE. (See Pipe, Cement Lined.)

CHEMICAL FEED APPARATUS.
Builders-Providence, Inc. (Divn. of Builders Iron Fdry.) xii
Omega Machine Co. v
% Proportioneers, Inc. % ..... Following front cover
Wallace & Tiernan Co., Inc. xx

CHLORINATORS.
Builders-Providence, Inc. (Divn. of Builders Iron Fdry.) xii
% Proportioneers, Inc. % ..... Following front cover
Wallace & Tiernan Co., Inc. xx

CLEANING WATER MAINS.
Flexible Underground Pipe Cleaning Co. xxii
National Water Main Cleaning Co. viii

COCKS, CURBS AND CORPORATIONS.
Eureka Cement Lined Pipe Co. xxvi
Hays Mfg. Co. xvii
Mueller Co. ..... Following front cover
Pierce-Perry Co. iii
Smith Mfg. Co., The A. P. ..... Following front cover

CONCRETE PIPE. (See Pipe, Concrete.)

CONSTRUCTION AIR TOOLS.
Edson Corp., The xxi

CONTRACTORS' EQUIPMENT.
Hydraulic Development Corp. x

CONTRACTORS.
Layne-New York Co. iii
Maher, D. L. Co. iii
Reppucci, C. & Sons iii

COUPLINGS, FLEXIBLE PIPE.
Johns-Manville Corp. ..... Following front cover

CURB BOXES.
Eureka Cement Lined Pipe Co. xxvi
Hays Mfg. Co. xvii
Mueller Co. ..... Following front cover
Pierce-Perry Co. iii

DIAPHRAGMS, PUMPS.
Edson Corp., The xxi
% Proportioneers, Inc. % ..... Following front cover

ENGINEERS.
Bailey, Howard E. ii
Barrows, H. K. ii
Camp, Dresser and McKee ii
Fay, Spofford and Thorndike ii
Haley and Ward ii
Knowles, Morris, Inc. ii
Metcalf and Eddy ii
Pirnie Engineers, Malcolm iii
Potts, Clyde ii
Weston and Sampson iii
Whitman and Howard iii

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued)*

ENGINES. (See Pumps and Pumping Engines.)

EQUIPMENT. (See Contractors' Equipment.)

FEED WATER FILTERS.
% Proportioneers, Inc. % ..... Following front cover
Ross Valve Mfg. Co., Inc. xxiii

FILTRATION PLANT EQUIPMENT.
Builders-Providence, Inc. (Divn. of Builders Iron Fdry.) xii
% Proportioneers, Inc. % ..... Following front cover

FLAP VALVES.
Eddy Valve Co xix

FLEXIBLE JOINTS.
U. S. Pipe and Foundry Co. xxxiv

FURNACES.
Hydraulic Development Corp. x
Leadite Co. ..... Back cover
Mueller Co. ..... Following front cover

GATE VALVES. (See Valves.)

HOSE, SUCTION AND CONDUCTION.
Edson Corp., The xxi

HYDRANTS, FIRE.
Eddy Valve Co. xix
Kennedy Valve Mfg. Co. xiii
Ludlow Valve Mfg. Co. xxviii
Mueller Co. ..... Following front cover
Rensselaer Valve Co. xxvii
Smith Mfg. Co., The A. P. ..... Following front cover
Wood, R. D., Co. ..... Following front cover

HYDRANTS, PUMPS.
Edson Corp., The xxi

LEAD.
National Lead Co. of Mass. xv

LEAD PIPE. (See Pipe, Lead.)

LEAD WOOL.
National Lead Co. of Mass. xv

LEAK FINDERS.
Edson Corp., The xxi

METERS, OIL AND WATER.
Badger Meter Mfg. Co. ..... Following front cover
Builders-Providence, Inc. (Divn. of Builders Iron Fdry.) xii
Hersey Mfg. Co. iv
Neptune Meter Co. vii
Pipe Founders Sales Corp. xxii
Pittsburgh Equitable Meter Divn. ..... Following front cover
Worthington-Gamon Meter Co. vi

METER COUPLINGS.
Badger Meter Mgf. Co. ..... Following front cover
Ford Meter Co. xvi
Hays Mfg. Co. xvii
Hersey Mfg. Co. iv
Mueller Co. ..... Following front cover
Neptune Meter Co. vii
Red Hed Mfg. Co. xxiv
Smith Mfg. Co., The A. P. ..... Following front cover
Worthington-Gamon Meter Co. vi

METERS (Venturi Type.)
Builders-Providence, Inc. (Divn. of Builders Iron Fdry.) xii

METER BOXES.
Ford Meter Co. xvi
Mueller Co. ..... Following front cover

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued)*

METER TESTERS.
Badger Meter Mfg. Co. ..... Following front cover
Ford Meter Co. xvi
Mueller Co. ..... Following front cover
Neptune Meter Co. vii

OIL ENGINES.
Edson Corp., The xxi
Fairbanks, Morse Co. xi

OZINATION EQUIPMENT.
Welsbach Corp. xxv

PIPE, ASBESTOS CEMENT.
Johns-Manville Corp. ..... Following front cover

PIPE, BRASS.
Pierce-Perry Co. iii

PIPE, CAST IRON (and Fittings.)
Builders-Providence, Inc. (Divn. of Builders Iron Fdry.) xii
Cast Iron Pipe Research Association xxxiii
Pipe Founders Sales Corp. xxii
U. S. Cast Iron Pipe and Foundry Co. xxxiv
Warren Foundry and Pipe Co. xxi
Wood, R. D. Co. ..... Following front cover

PIPE, CEMENT LINED.
Cast Iron Pipe Research Association xxxiii
Cement Lined Pipe Co. iii
Eureka Cement Lined Pipe Co. xxv
Pipe Founders Sales Corp. xxii
U. S. Pipe and Foundry Co. xxxiv

PIPE, COATING AND LININGS.
Barrett Division ..... Following front cover
Centriline Corp. ..... Following front cover
Koppers Company, Inc. ..... Following front cover

PIPE, CONCRETE.
Lock Joint Pipe Co. ..... Facing front cover

PIPE CUTTING MACHINES.
Smith Mfg. Co., The A. P. ..... Following front cover

PIPE JOINTING MATERIAL.
Atlas Mineral Products Co. xxvi
Hydraulic Development Corp. x
Leadite Co., The ..... Back cover
National Lead Co. of Mass. xv

PIPE, LEAD.
National Lead Co. of Mass. xv
Pierce-Perry Co. iii
Red Hed Mfg. Co. xxiv

PIPE LINING.
Centriline Corp. ..... Following front cover

PIPE, PRESTRESSED CONCRETE.
Lock Joint Pipe Co. ..... Facing front cover

PIPE, WROUGHT IRON AND STEEL.
Pierce-Perry Co. iii

PLUG VALVES.
Eddy Valve Co xix
Hays Mfg. Co. xvii
Mueller Co. ..... Following front cover
Pittsburgh Equitable Meter Divn. ..... Following front cover

PITOMETERS.
Pitometer Co., Engrs. iii

PORTABLE AIR COMPRESSORS. (See Air Compressors.)

PRESSURE REGULATORS.
Mueller Co. ..... Following front cover
Pittsburgh Equitable Meter Divn. ..... Following front cover
Ross Valve Mfg. Co., Inc. xxiii

PROVERS, WATER.
Badger Meter Mfg. Co. ..... Following front cover

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued)*

PUMPS AND PUMPING MACHINES.
Edson Corp., The xxi
Fairbanks, Morse and Co. xi
Layne-New York Co., Inc. iii
Maher Co., D. L. iii
Morris Machine Works xviii
Ross Valve Mfg. Co., Inc. xxiii
Turbine Equipment Co. xiv

RATE CONTROLLERS AND GAUGES.
Badger Meter Mfg. Co. ..... Following front cover
Builders-Providence, Inc. (Divn. of Builders Iron Fdry.) xii

SHEAR GATES.
Eddy Valve Co xix
Mueller Co. ..... Following front cover

SLEEVES AND VALVE TAPPINGS.
Eddy Valve Co xix
Ludlow Valve Mfg. Co. xxviii
Mueller Co. ..... Following front cover
Rensselaer Valve Co. xxvii
Smith Mfg. Co., The A. P. ..... Following front cover

SUPPLIES AND TOOLS.
Hydraulic Development Corp. x
Leadite Co., The ..... Back cover
Mueller Co. ..... Following front cover
Pierce-Perry Co. iii

TANKS, WATER, FUEL, SLUDGE DIGESTION.
Preload Enterprises, Inc. ..... Following front cover

TAPPING MACHINES.
Hays Mfg. Co. xvii
Mueller Co. ..... Following front cover
Red Hed Mfg. Co. xxiv
Smith Mfg. Co., The A. P. ..... Following front cover

TAPPING SLEEVES. (See Sleeves and Valves, Tapping.)

VALVE BOXES.
Eddy Valve Co xix
Kennedy Valve Mfg. Co. xiii
Mueller Co. ..... Following front cover
Pierce-Perry Co. iii
Pipe Founders Sales Corp. xxii
Smith Mfg. Co., The A. P. ..... Following front cover
Wood, R. D. Co. ..... Following front cover

VALVE INSERTING MACHINES.
Mueller Co. ..... Following front cover
Smith Mfg. Co., The A. P. ..... Following front cover

VALVES, CHLORINE.
Wallace and Tiernan Co., Inc. xx

VALVES, GATE.
Eddy Valve Co xix
Kennedy Valve Mfg. Co. xiii
Mueller Co. ..... Following front cover
Pierce-Perry Co. iii
Rensselaer Valve Co. xxvii
Smith Mfg. Co., The A. P. ..... Following front cover
Wood, R. D. Co. ..... Following front cover

VALVES, REGULATING.
Mueller Co. ..... Following front cover
Ross Valve Mfg. Co., Inc. xiv

WATER WASTE DETECTION.
Pitometer Co., The iii

WELL CONNECTIONS.
Red Hed Mfg. Co. xxiv

WELLS, GRAVEL, FILTER AND DRIVEN.
Edward F. Hughes Company iii
Maher Co., The D. L. iii

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